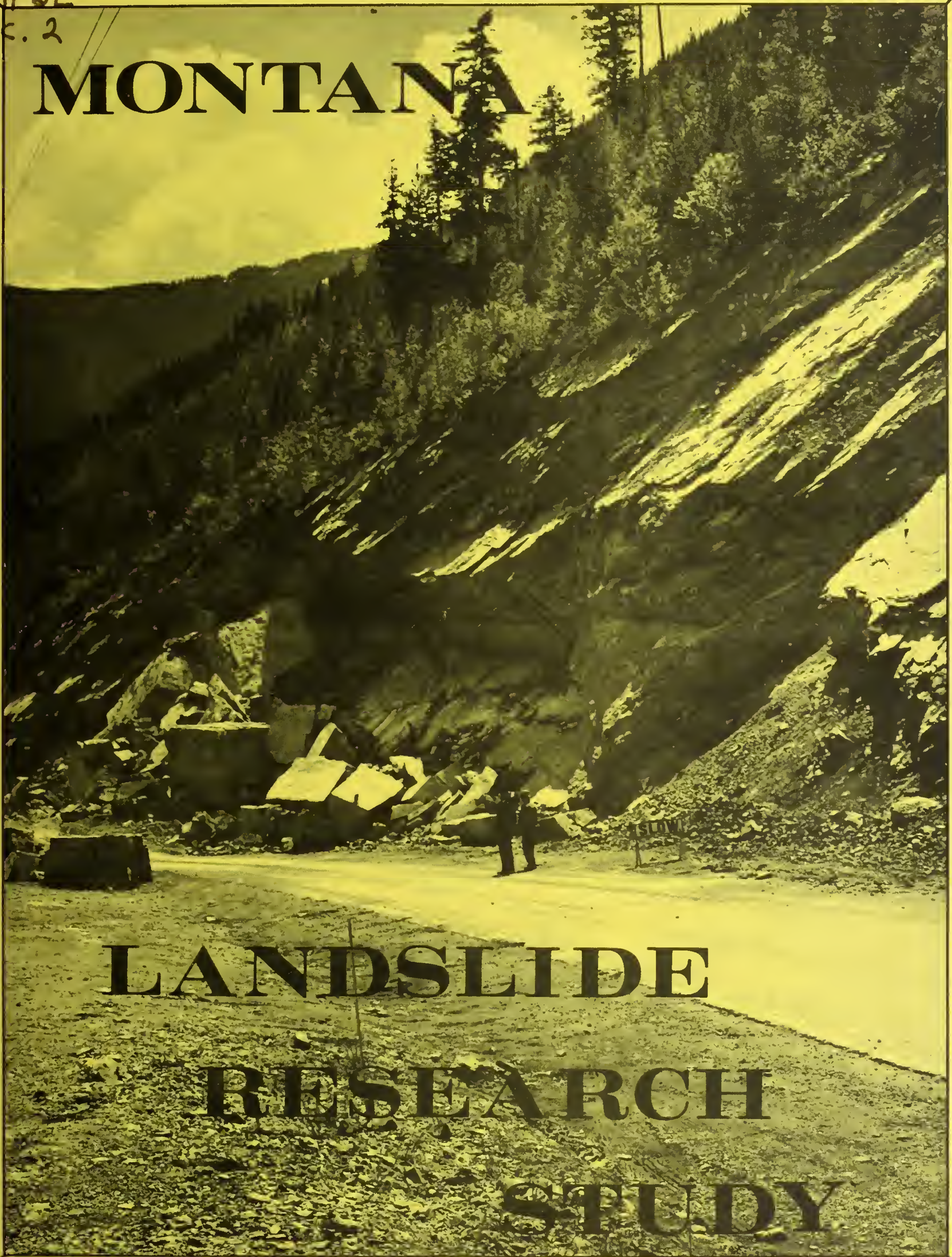
S
551.35
H3L
c.2

STATE DOCUMENTS dup

MONTANA

LANDSLIDE RESEARCH STUDY

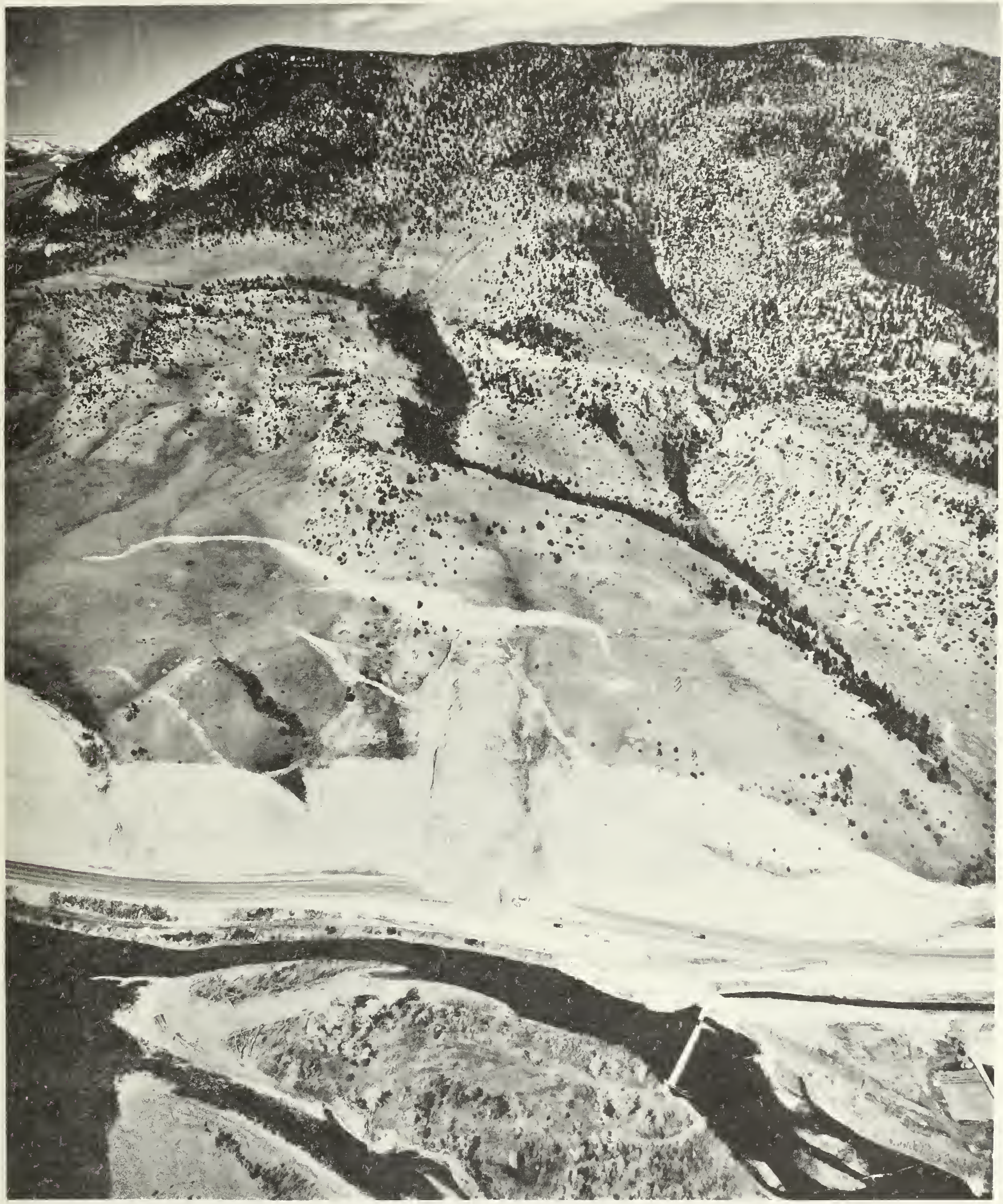


862

Montana State Library
3 0864 1006 4564 0

Digitized by the Internet Archive
in 2013

http://archive.org/details/landslideresearc00will



Typical Montana Landslide

Livingston Landslide, May 16, 1967, shortly after latest recurrence.

LANDSLIDE RESEARCH

by

Materials Division

Dennis A. Williams, Chief Geologist
Charlie H. Clark, Principal Investigator

Prepared for

Montana Highway Commission
Planning Survey Section

In cooperation with

U.S. Department of Transportation
Federal Highway Administration
Bureau of Public Roads

" The opinions, findings and conclusions expressed in this publication are those of the authors and not necessarily those of the Bureau of Public Roads."

ACKNOWLEDGEMENTS

The personnel of the Geology Section of the Montana State Highway Commission wish to express their sincere appreciation to all the personnel within the Montana State Highway Commission and the United States Bureau of Public Roads for their cooperation in the conduct of this investigation.

Special appreciation to Mr. J. R. Sallberg, Structures and Applied Mechanics Division of the Bureau of Public Roads for his advice and constructive criticism.

TABLE OF CONTENTS

Page

LIST OF ILLUSTRATIONS v

LIST OF TABLES vi

ABSTRACT vii

(1) INTRODUCTION 1

(2) METHODS AND EQUIPMENT 1

(3) DESCRIPTIONS OF LANDSLIDE AREAS 4

DETAILED LANDSLIDE STUDIES 11

Armells Creek Slide (No. A-1) 11

Belt Slide (No. A-2) 19

East Glacier Slide (No. A-3) 29

Frenchtown Slide (No. A-4) 34

Flathead Slide (No. A-5) 39

Garrison-Avon Slide (No. A-6) 42

Georgetown Lake Slide (No. A-7) 50

Havre Slide (No. A-8) 54

Ovando Slide (No. A-9) 61

Livingston Slide (No. A-10) 67

Pipe Organ Slide (No. A-11) 89

Ballantine Slide (No. A-12) 101

(4) CONCLUSIONS 107

(5) FURTHER RESEARCH RECOMMENDATIONS 108

(6) REFERENCES 109

LIST OF ILLUSTRATIONS

Rockslide in Flathead County, Slide B .... Cover Page

Frontpiece i

Armells Creek Slide (No. A-1) 12
14
17
18

Belt Slide (No. A-2) 20
22
26

East Glacier Slide (No. A-3) 30

Frenchtown Slide (No. A-4) 38

Garrison-Avon Slide (A-6) 43

Georgetown Slide (A-7) 51

Havre Slide (A-8) 55
58

Ovando Slide (A-9) 62
64
66

Livingston Slide (A-10) 68
69
70
71
72
73
74
75
76
77
86
87
88

Pipe Organ Slide (A-11) 90
91
94
97
98
99
100

Ballantine Slide (A-12) 104
105

LIST OF TABLES

Page
1. Landslide Classification 8
9
10

ABSTRACT

The Montana Highway Commission, in cooperation with the United States Department of Transportation, Bureau of Public Roads, undertook a Highway Research and Planning Program in 1963. The project was initiated for the purpose of studying the known landslides on and near existing or planned highways in Montana, and compiling a comprehensive file of unstable areas for future reference.

Equipment and technique utilized in this investigation were those in current use, including surveys, drilling, portable geophysical equipment and various type of aerial photography. Analysis was made using conventional shear arc and glide plane computation. Compilation of data obtained is summarized in this report with emphasis on several of the larger and/or more complex failures.

The program has succeeded in cataloging and filing of many failures in detail. This will be useful for reference in future highway locations and construction. Certain areas and geological settings in Montana have been dicovered which are very susceptable to instability.

The need for better tools and techniques in the study of unstable or potentially unstable area is discussed in the last portion of this report.

FINAL REPORT

LANDSLIDE RESEARCH

H.P.R.-1 (6)

Project No. 0026-10-7908-9602

(1) INTRODUCTION

The objectives of this study at its inception in 1963 were as follows:

A. Study of all known active and quiescent slides and unstable areas along Montana Highway systems.

B. Compilation of the data gained and formation of a comprehensive library of the landslide areas. This library would be useful as a reference to aid in landslide recognition and subsequent avoidance of unstable areas.

The scope of the individual investigations included geologic study, subsurface investigation, topographic mapping and sample testing at selected landslides as well as the statewide search for all unstable areas. Compilation of all of these data was not completed and will never be completed as new failures are constantly occurring, but many representative landslides were studied in some detail. Most of the information in this report was taken from project files which form part of the aforementioned library.

(2) METHODS AND EQUIPMENT

Most small failures were investigated by simple methods. Several landslides were investigated with more sophisticated equipment including drills, refractive seismograph, resistivity equipment, aerial photography and subsequent topographic mapping, controlled surveys of movement, and laboratory testing of samples with completed shear analysis. The investigation usually included the following steps:

A. Recognition of the landslide.

B. Preliminary survey with tape and compass, location of head and toe, geologic mapping of area, location of seeps and/or springs and configuration of the slide plane where possible.

C. Compilation of data which either was introduced into the files or became the basis for justification of further investigation.

The equipment and techniques utilized in this research project are those in general use. All methods and instruments used have limitations and all were most useful when augemented by others. To help evaluate these the following discussion is presented:

(1) Uncomplicated landslides could often be evaluated by simple methods. For example; if the failure occurred in hard rock it was almost always controlled by bedding or jointing attitude. These rock features were rapidly determined with a Brunton Compass and a little geological information. If the failure was in non-cohesive materials such as sand or gravel, it was usually a simple task to trace the ground and surface water and evaluate the shearing resistance of the materials with simple tests.

(2) Complicated landslides usually required a much more sophisticated approach. This usually included the use of extensive drilling, geophysical study, with more intensive shear testing as well as mapping, etc.

The equipment used in these investigations and pertinent comments are presented herein.

A. Drilling Equipment. The Montana Highway Department has five core drilling rigs, one horizontal drill, one churn drill and several truck mounted augers. With this equipment samples of all types and sizes up to and including four-inch diameter were obtained. To locate slide planes, one to two inch diameter perforated plastic pipe was placed in completed test holes and the intervening space between the test hole wall and pipe backfilled with sand or fine gravel. Probing this pipe with an elongated

metal probe at regular time intervals soon disclosed the approximate location of the sliding surface if appreciable movement had occurred. This method was satisfactory in most cases.

B. Testing Equipment: The central laboratory of the Montana Highway Department is equipped with triaxial shear testing, direct shear testing, and consolidation testing machines. This equipement performed very satisfactorily in all cases although the interpretation of test data and relationship of test results to actual field conditions was often very difficult to assess.

C. Geophysical Equipment: A 12-channel refraction seismograph was utilized to good advantage in several landslide studies. It is best used in extending the subsurface data obtained in drilling and in our opinion, can be used to its best advantage if drilling data is available and if the general geologic setting is known.

Earth resistivity equipment was utilized in several investigations. This has proven to be a useful instrument by many investigations, however, it was not given sufficient use during this program to evaluate the applicability of the instrument for landslide study.

D. Aerial Photography: Stereoscopic, vertical photos were used extensively for mapping and geologic structure interpretation and one landslide was photographed in color. Study of these color photographs indicated the usefulness of this method in determination of the geologic setting and spring areas. Further study of this tool for landslide investigation should be made.

The stability analysis of failure areas utilized the commonly accepted circular arc or block glide analysis using seepage forces where applicable. The circular arc analysis is programmed for computer solution. This approach seemed adequate in all cases.

(3) DESCRIPTIONS OF LANDSLIDE AREAS

The sites studied were statewide, however, several landslides that have been studied in detail are described in this report as typical of the types of landslide and investigative methods used as well as corrective action attempted or recommended. The detailed analysis of the following slides are included in this report.

Landslide A-1 (Armells Creek Slide)

Block glide slide at or near the contact between Bearpaw shale and the underlying Judith River Sandstone described on page 11. This landslide occurred during construction of the primary roadway designated as F-333. The original movement was fairly rapid and precipitated an intensive investigation. Toe loading was unsuccessful as were horizontal drains and drainage trenches in stabilizing the slide, although the rate of movement has been substantially reduced.

Landslide A-2 (Belt Fill Slide)

Rotational slide failure in a high fill section, described in detail on page 19. This failure occurred about 10 years after fill completion following a very wet spring. The first attempt to stabilize the fill with a counterbalance berm and interior drainage showed signs of failure during construction. A redesign for this project has been rebuilt with more extensive counterbalances and better subsurface drainage. The fill has been stable since this remedial action.

Landslide A-3 (East Glacier Slide)

Block glide slide at or near the contact between glacial till and Colorado shale, described in detail on page 29. This failure

occurred at intermittent time intervals and created a constant maintenance problem. Several attempts to stabilize the landslide through partial head removal and with horizontal drains to eliminate ground water were only partially successful. This landslide was finally corrected by complete removal of all unstable materials, and has remained stable to date.

Landslide A-4 (Frenchtown Slide)

Block Glide slide at or near the contact between Pleistocene lake bed sediments and the underlying cemented gravel. See page 34 for detailed description. This landslide failed toward the roadway at intermittent time intervals. A rather complete investigation was accomplished and the information was utilized to design a new highway in the area. Attempts to stabilize the slide were not made in-as-much as new construction will avoid immediate vicinity.

Landslide A-5 (Flathead Slide)

Block Glide slide on joint and bedding planes in the rock with slumping of the overlying glacial till. A more detailed description can be found on page 39. This slide was precipitated by attempting to make a large cut through materials with steeper cut slopes than the geological setting would allow. The solution was to re-design the cut slope to conform with the bedding and joint attitudes of the rock and slope stability of overlying glacial till.

Landslide A-6 (Garrison-Avon Slide)

Block Glide slide on bedding plane, described in detail on page 42. This landslide failed quite rapidly in four movements between 1958 and 1963. One attempt was made to remove driving weight without success. The second removal of head weight has apparently resulted in a stable condition.

Landslide A-7 (Georgetown Lake Slide)

Block Glide slide on rock bedding and joint planes, described in detail on page 50. This failure occurred in solid rock within a very steep cutslope and is controlled by bedding and jointing attitudes. Corrective measures have been confined to removal of the materials as they slide onto the highway.

Landslide A-8 (Havre Slide No. 1 & 2)

Rotational failures in soft silt, described on page 54. These failures occurred near the same location and are both in the same materials. The primary cause of both was groundwater and the corrective measures of establishing drainage were effective in both cases.

* Landslide A-9 (Ovando Slide)

Rotational slide in soft clay and silt, described on page 61. This failure occurred on a hillside overlooking the Blackfoot River and moved shortly after highway construction. The shear plane was found by the use of resistivity equipment. The roadway has been stabilized by grade and alignment adjustment.

*Landslide A-10 (Livingston Slide)

Rotational slide in toe of pre-existing large landslide described on page 67. A portion of the toe of a pre-existing, large landslide was cut during highway construction and a small portion of the landslide was reactivated. Removal of a part of the head portion of the failure temporarily stabilized the area. During 1966 further small movements were noted and the landslide investigated with Highway Planning and Research funds. In 1967 a second rapid failure occurred and the area is evidently in a temporary state of equilibrium at the present time. (see frontpiece).

Landslide A-11 (Pipe Organ Slide)

Combination rotational and block glide slide within a pre-existing slide, described on page 89. A very large pre-existing landslide was re-activated by highway construction and failed in several discreet blocks, which blend together into a planar slide which resembles a block glide. The movement at this slide was very constant with an average rate of 0.25 feet per week. The removal of ground water has temporarily contained the failure.

Landslide A-12 (Ballantine Slide)

Rotational landslide in weathered Bearpaw shale, described on page 101. New highway construction caused the reactiviation of a pre-existing landslide. The area was stabilized with horizontal drains.

---

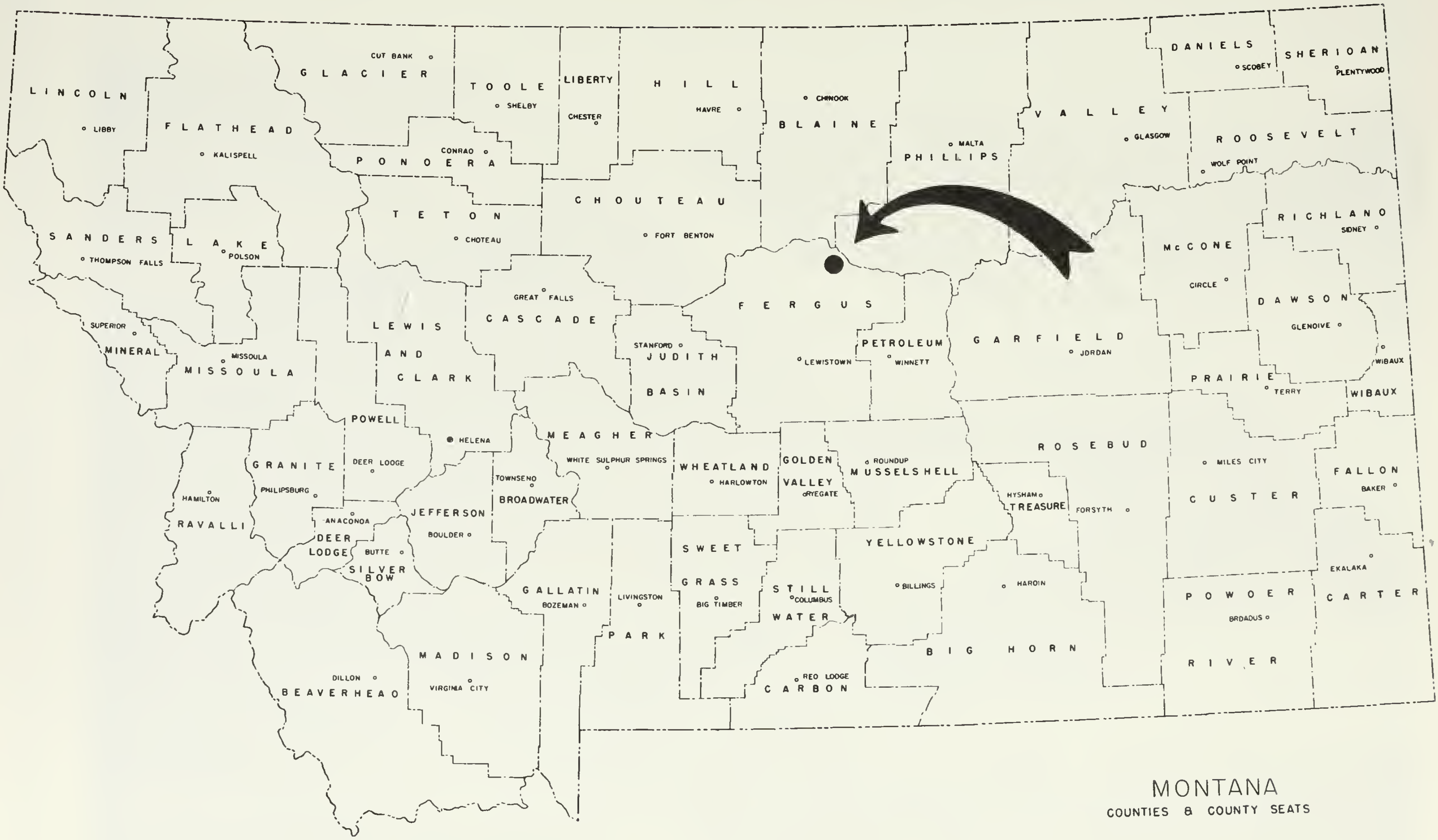
* Slide investigation accomplished with Highway Planning and Research funds.

Slide No.	County	Type Failure	Geologic Materials Description	Brief Description of Unstable Area
A-11	Beaverhead	Rotational	Volcanic Debris	See detailed description, page 89
1	Big Horn	Planar	Cody Shale	Large fill over unstable hillside, with seepage and springs evident.
2	Cascade	Planar along bedding plane	Cretaceous sandstone and shale	Natural unstable area with excessive ground water.
3	Cascade	Rotational in cutslope	Colorado Shale	Glacial till over gray, fissile clay shale and excessive ground water.
4	Cascade	Rotational in cut slopes	Morrison formation	Several slides in natural slopes and in cut slopes.
5	Cascade	Rotational	Jurassic sequence	Very large landslide, not affecting the highway system.
A-2	Cascade	Part rotational, part planar	Morrison formation	See detailed description, page 19.
6	Chouteau	Rotational	Glacial till over shale	Several slides caused by fill construction on unstable hillside.
7	Dawson	Rotational	Ft. Union formation	Many landslides along the south side of the Yellowstone River.
A-1	Fergus	Part rotational, part planar	Bearpaw formation	See detailed description, page 11.
8	Flathead	Rockfall	Piegan Gp. Pre-Cambrian	Bedding attitude unfavorable for cut slope design.
A-5	Flathead	Rockfall	Piegan Gp. Pre-Cambrian	See detailed description, page 39.

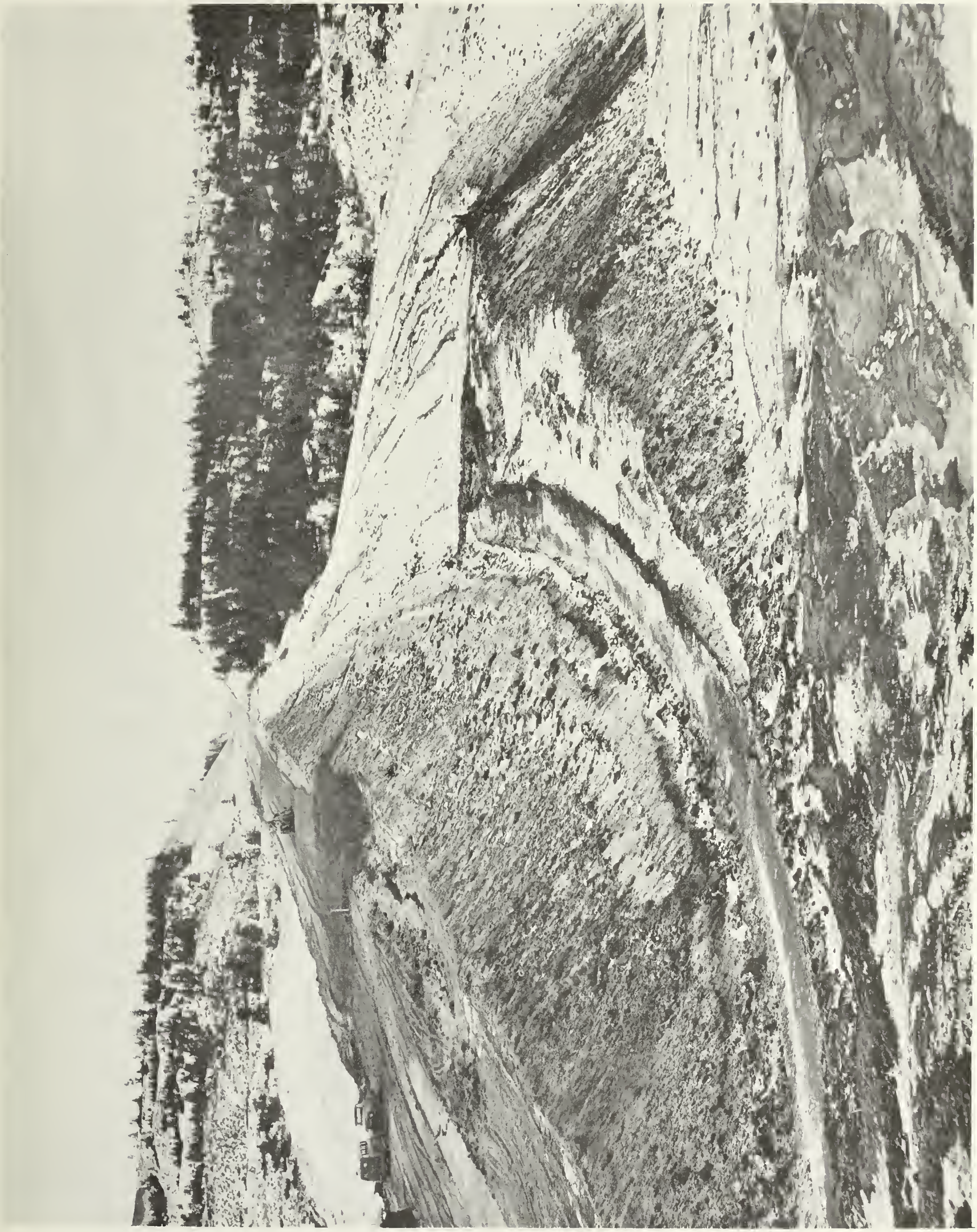
Slide No.	County	Type Failure	Geologic Materials Description	Brief Description of Unstable Area
9	Gallatin	Part rotational, part rockfall	Devonian and Mississippian	Shale and limestone shattered between at least two major faults.
10	Gallatin	Rotational	Volcanic rock alluvium	Fine grained alluvial debris at mouth of Canyon - Cut slope failure.
A-3	Glacier	Planar	Glacial till over shale	See detailed description, page 29.
11	Glacier	Fill failure	Glacial till over shale	Fill constructed on till with excessive ground water present.
12	Glacier	Rotational	Glacial till & lake beds	Failure in natural slope, toe cut out during construction of Channel Change.
A-7	Granite	Rockfall	Pre-Cambrian Argillite	See detailed description, page 50.
A-8	Hill	Rotational	Bearpaw formation	See detailed description, page 54.
13	Judith Basin	Rotational	Morrison formation	Several slides in natural and highway cut slopes.
14	Lewis & Clark	Part mudflow, part rotational	Two Medicine formation	Slide caused by fill construction on hillside with excessive ground water.
15	Lewis & Clark	Part Rotational, part planar	Tertiary Volcanic rubble zone	Large slide caused by cutting off part of "toe" area with highway cut.
16	Lincoln	Debris and Rockfall	Alluvium and Pre-Cambrian	Steep sidehills with heavy rains caused failures.
17	Meagher	Rotational	Alluvial & decomposed shale	Several rotational slides on natural hillsides and cut slopes - excessive water.
18	Missoula	Rotational	Lacustrine varved sediments on hillside	Natural drainage on sidehill blocked by highway construction.
A-4	Missoula	Planar	Lacustrine varved sediments over conglomerate	See detailed description, page 34.

Slide No.	County	Type Failure	Geologic Materials Description	Brief Description of Unstable Area
19	Missoula	Debris & rock-fall	Pre-Cambrian rock & colluvium	Highway construction undercut natural Talus slope attitude.
A-10	Park	Part planar, part rotational	Entire Paleozoic Section	See detailed description, page 67.
20	Phillips	Rotational	Bearpaw formation	Several cut slope failures, clay, shale, excessive ground water.
A-6	Powell	Planar	Colorado Shale	See detailed description, page 42.
A-9	Powell	Rotational	Alluvium	See detailed description, page 61.
21	Powell	Rotational	Alluvium	Cut slope failure on sidehill-excessive ground water
22	Prairie	Rotational	Ft. Union formation	Foundation failure under high fill.
23	Sanders	Rotational	Lacustrine varved sediments	Sidehill failure after fill construction-excessive ground water.
24	Sanders	Rockfall	Colluvium & Talus	Highway construction undercut talus slopes.
25	Sanders	Rotational	Lacustrine varved sediments	Cut slope failure following heavy rains.
26	Stillwater	Planar	Rubble zone	Sidehill fill failed with slope following heavy rainfall.
27	Sweetgrass	Unknown	Volcanic rock & detritus	Fill failures following heavy rainfall.
28	Sweetgrass	Unknown	Jurassic and Cretaceous rock	Very large active landslide traversed by highway.
29	Toole	Rotational	Glacial till over shale	Several landslides caused by excessive ground water.
A-12	Yellowstone	Rotational	Bearpaw formation	Highway construction reactivated on old slide, See detailed description, page 101.

ARMELLS CREEK LANDSLIDE



SLIDE NUMBER A-1



Taken in 1959 showing small failure and part of the berm constructed below the fill. Major portion of the slide is toward the upper left.

Plate No. 1

Landslide No. A-1

Armells Creek
LANDSLIDE EVALUATION FORM

1. Location County Fergus Township 21N Range 23E Sec. 12
Highway U.S. 191 Project No. F-333 Station 1030± to 1080±

2. Dates of study June, 1955 to Jan. 1962

Remarks concerning general weather and rainfall/snowfall conditions during the time of study.

Summers usually semi-arid, winters usually have a moderate snow fall.

Actually seasonal rainfall has not been conclusively proven to be related to sliding.

3. Date of origin Pre-highway Date Movement Ceased Still active

Most active season Spring and summer 1955

Relative activity (speed, etc.) from origin to present The original movement was fairly rapid, however, subsequent movements have been slower. Measurement from January 1959 to Sept. 1961 indicate a movement of 15.6 feet at right angles to the highway.

4. Physical dimensions The total slide covers approximately 20 acres in lateral extent.

Approximate Yardage involved The yardage involved has not been determined but is estimated to be well in excess of 1,000,000 cubic yards.

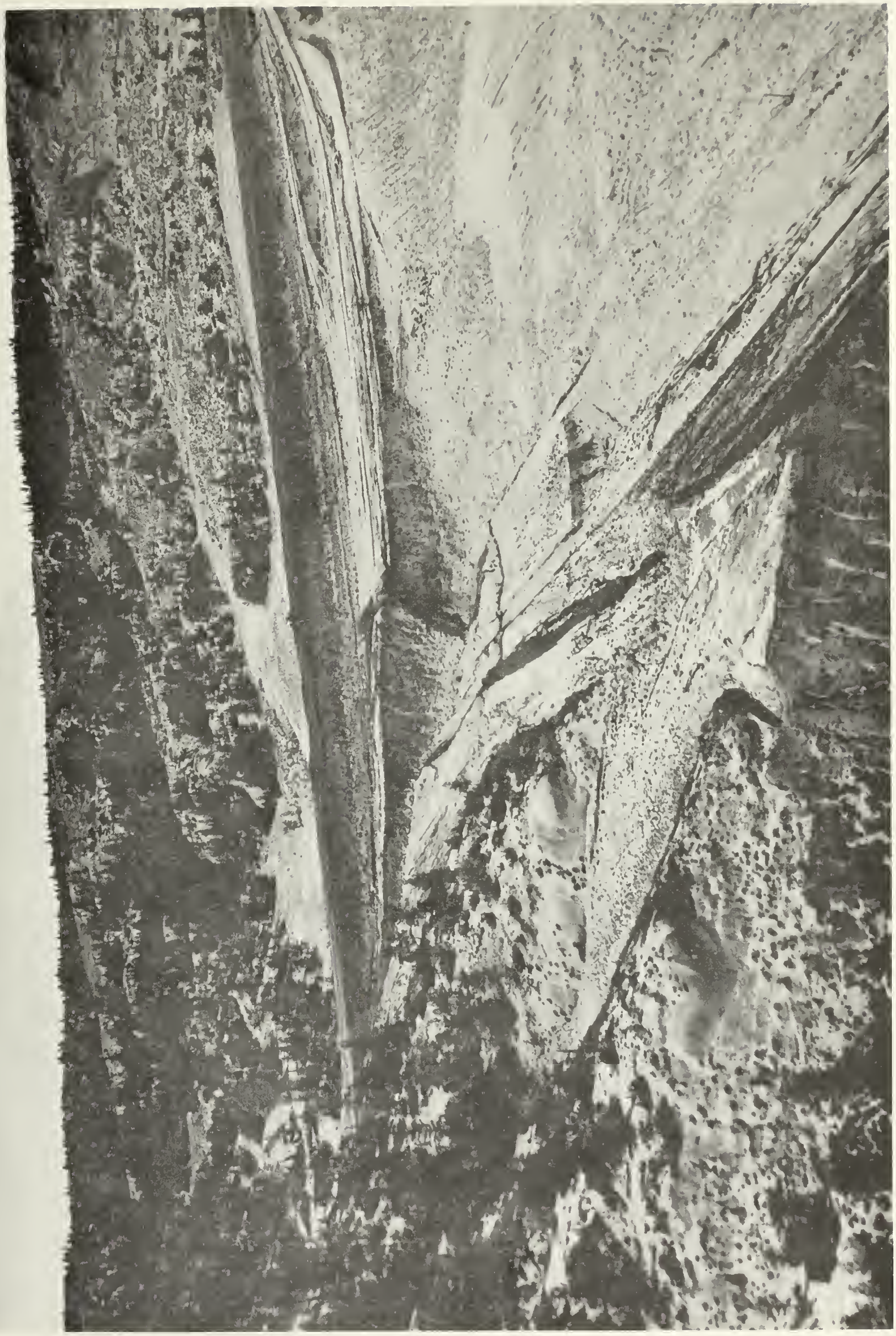
Proposed fill slope at 2:1 with height of 90' ±. Failed when approximately one half completed.

5. Water table, rainfall information, and remarks concerning the effect of water in the unstable condition The initial movement triggered by the fill placement was probably related to changes in the hydrostic head caused by the fill weight.

The ground water movement has since become very complex for various reasons.

6. Description of rock or soil types involved in the slide and in the adjacent area

The slide plane is in or at the base of the Bearpaw Shale which is a bentonitic marine shale of Upper Cretaceous age. The shale is underlain by the Judith River Sandstone, also Upper Cretaceous in age, which is a soft, friable, arkosic massive sandstone. (See photos on pages 17 and 18 for cut slope failures in this area.)



View looking toward the slide. The headwall scarp is just below the horizon. The toe is near the hummocky ground in left, lower portion. Note hummocky ground, poor drainage, start of counterbalance berm, etc.

Plate No. 2

F-333 (Armells Creek)

7. Investigation and Cause: This landslide has been investigated by several Montana Highway Geologists and by the Oregon State Highway Department, acting in a consulting capacity. The investigations have included 147 vertical test holes, 15 horizontal drainage holes, seismic surveys, geological study and study by men conversant with hydraulics, soil mechanics, etc.

These investigations have succeeded in locating the slide plane, determining shear values for the materials involved as well as locating ground water and related natural conditions.

During pre-highway time, several related large landslides developed either through erosion at the bases of the then existing slopes or by changes in ground water level with a resultant redistribution of weights and shearing strengths. (It is apparent that stream erosion of the Bearpaw Shale all along the Missouri River combined with mass sliding is the normal cycle of topographic development in this area.) During construction, one of the higher fill sections to be placed was over one of the slide blocks. The foundation failed when the fill was approximately one half completed. (At a height of 45± feet or ½ of the 90± feet planned.) The foundation failed because of a combination of the following two causes.

(1) Fill weight added to the pre-existing weak slide zone. *

(2) Fill weight partly blocking the pre-existing natural drainage through the slide, causing hydrostatic buildup within the fill foundation and the upstream side.

* Shear strengths of remolded Bearpaw Shale as determined by the Corps of Engineers at Fort Peck, Montana are as follows:

Angle of Internal Friction = $10^\circ - 10'$ to $16^\circ - 23'$

Cohesion 0 to 600 pounds per square foot.

Activiation of the small block directly under the fill apparently reactivated much of the old slide and created a very unstable area.

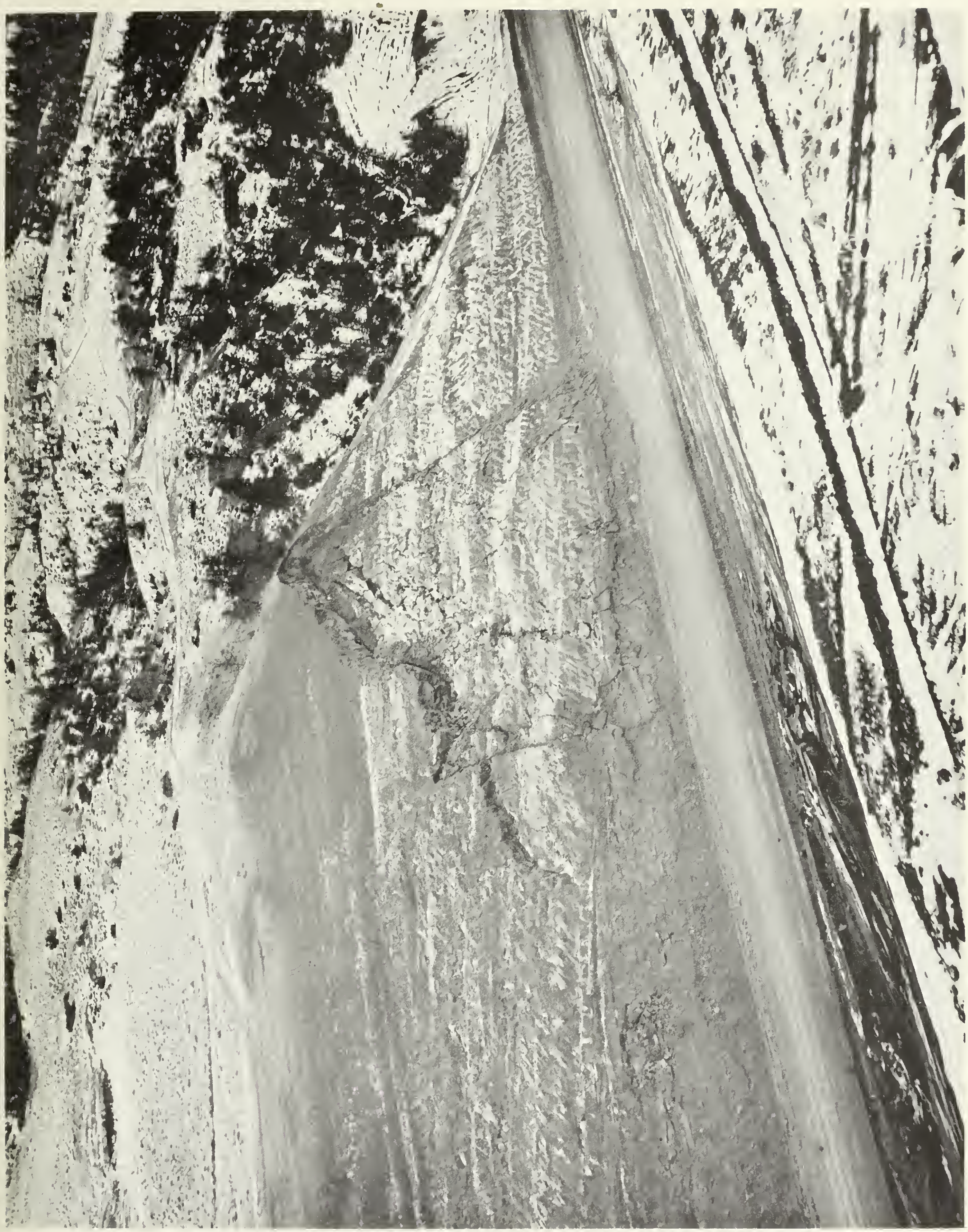
Although placement of a large berm has partly stabilized the fill and the highway is open for traffic; subsequent movement occurs each year requiring further roadbed repairs.

8. Corrective Measures: To date the corrective measures have included horizontal drain holes, a deep T-drain above the fill section drained by an electric pump, placement of a large berm on the downstream side of the fill, attempts to unload the slide head, and numerous roadway repairs. The applied corrective measures have successfully maintained the existing roadway. Movement has continued but only at a very slow rate so that with frequent roadbed repairs, a travelway has been maintained. The slide itself has not been cured and the movement may speed up or stop at any time. It is evidently in a state of near equilibrium at present and hopefully may continue to remain so. The only permanent solutions are as follows:

   (1) Remove all the slide material and the shale to a firm foundation of sandstone, (a measure which is deemed economically infeasible).

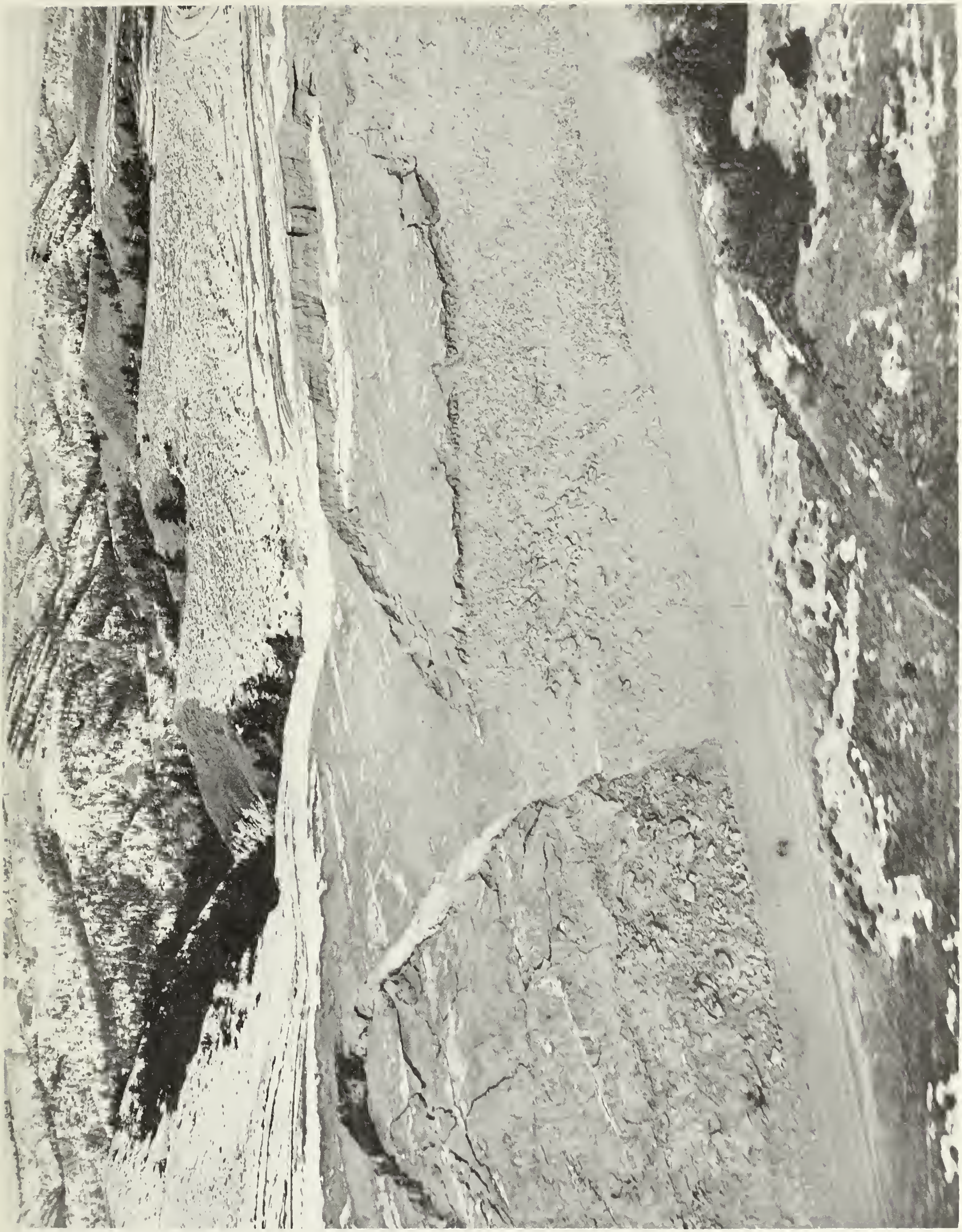
   (2) Move the roadway away from the slide area.

9. Data for avoiding similar situations in the future: This slide was recognized as a possible problem during the preliminary survey but was considered to be ancient and static. Any slide should be studied in considerable detail before it can be safely called stable. Highway location should avoid any area of questionable stability.



Failure in cutslope near Armells Creek in Bearpaw Shale.

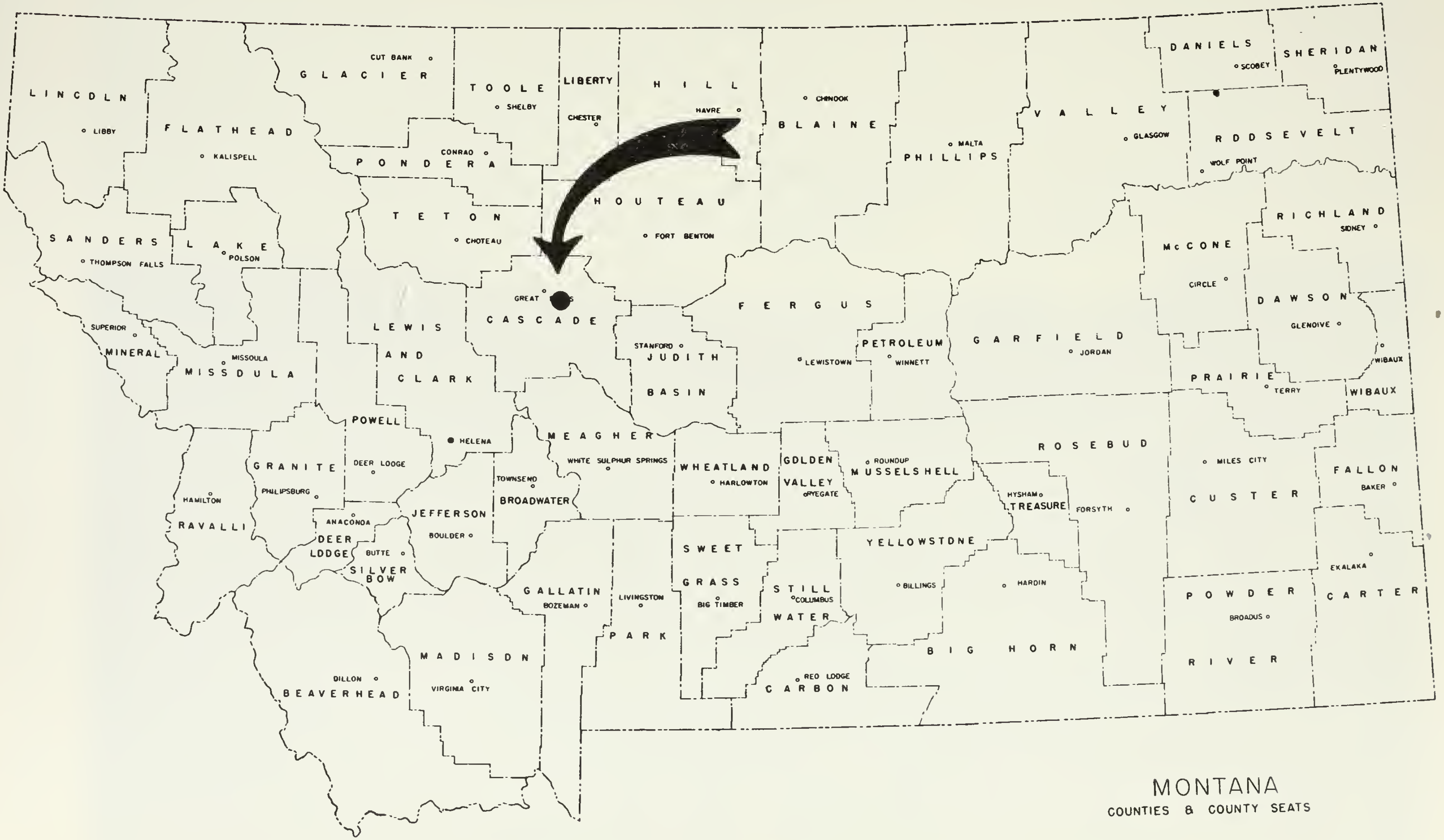
Plate No. 3



Failure in cutslope near landslide. Material is Bearpaw Shale cut on $1\frac{1}{2}$:1 slope.

Plate No. 4

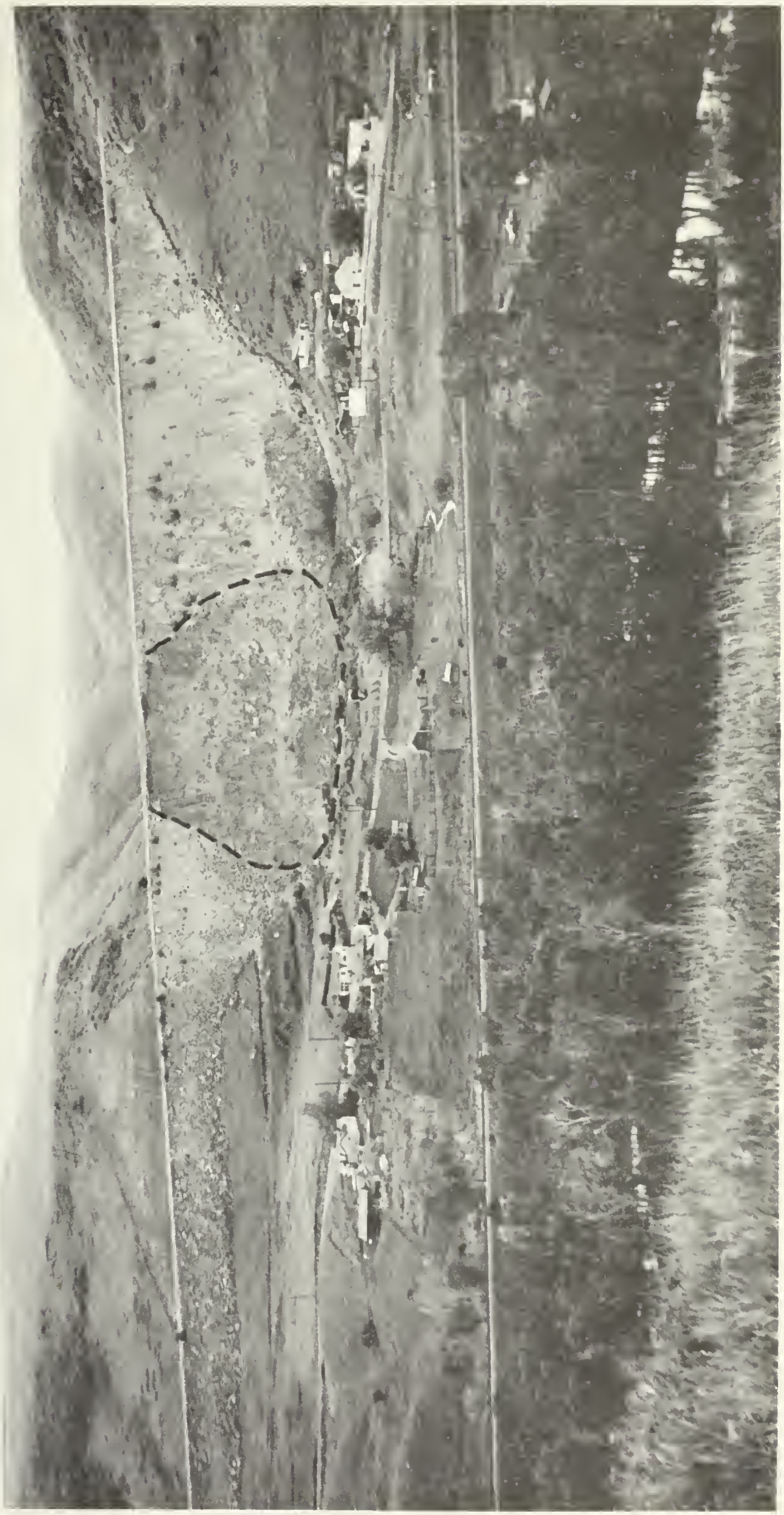
BELT LANDSLIDE



MONTANA

COUNTIES & COUNTY SEATS

SLIDE NUMBER A-2



Belt Slide

Note - downstream side of fill showing 1st failure in 1964; failure outlined with dashed lines.

Plate No. 2

Landslide No. A-2

Belt Slide

LANDSLIDE EVALUATION FORM

1. Location County Cascade Township 19N Range 6E Sec. 27
One half mile from Belt.

Highway U.S. 87 Project No. F 64(8) Station 800± 820±

2. Dates of study May and June, 1964, October to December 1965.

Remarks concerning general weather and rainfall/snowfall conditions during the time of study.

Overaverage rain prior to the failure in 1964, approximately 1.7" preceded this original failure.

3. Date of origin Fill placed in 1954 April 27, 1964 Date Movement Ceased Unknown

Most active season April, 1964

Relative activity (speed, etc) from origin to present Original Failure was very rapid, total movement occurred within minutes. The excessive strain which resulted in compressive cracking during the construction of the fill was stopped by the immediate placement of additional berm material to counterbalance the mass.

See plates No. 1, 2 & 3, pages 20, 22, and 26.

4. Physical dimensions

Approximate yardage involved In 1964, approximately 25,000 cubic yards
In 1965 approximately 65,000 cubic yards

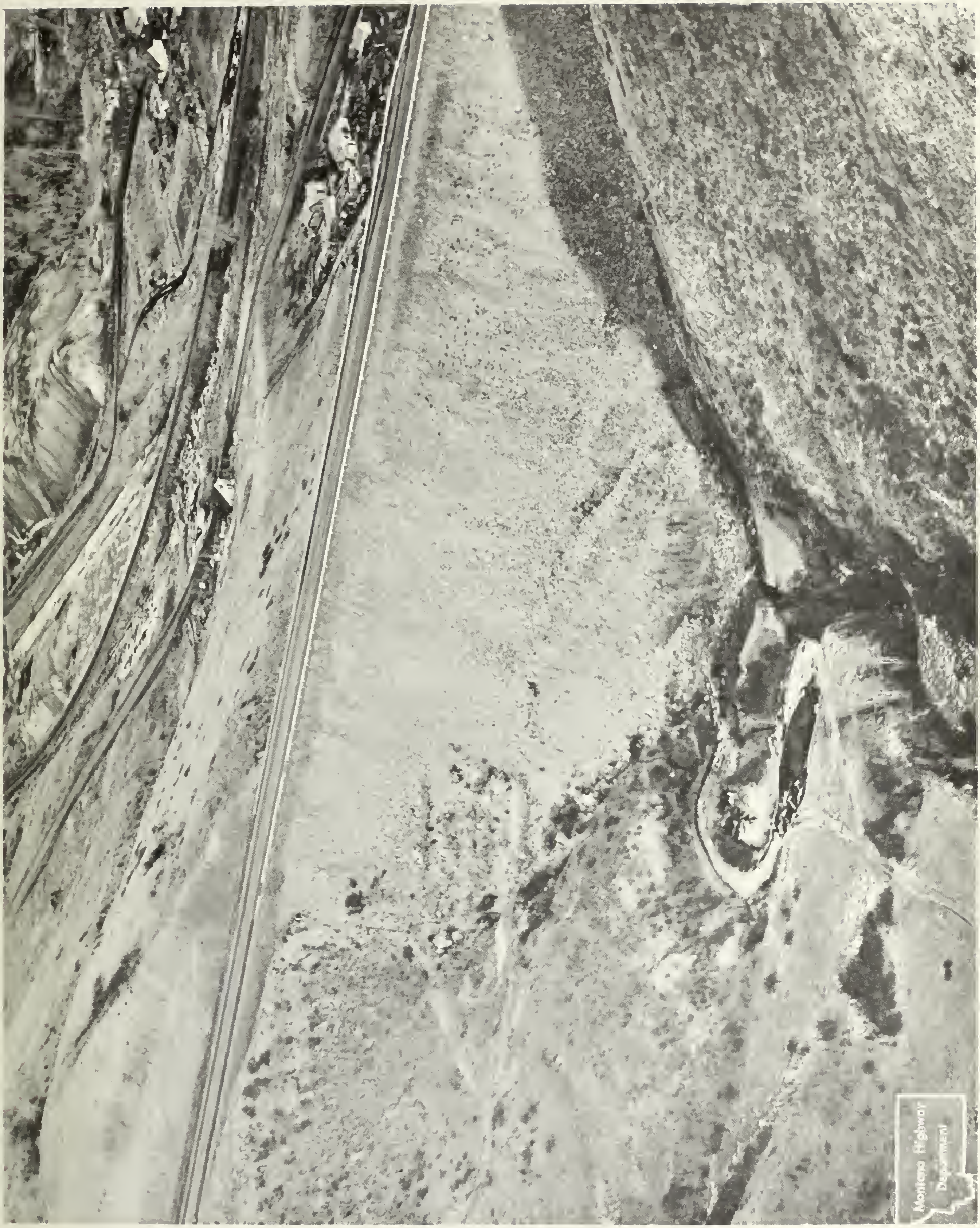
Height of fill slope 200 feet

Angle of fill slope 1½ to 1

5. Water table, rainfall information, and remarks concerning the effect of water in the unstable condition Heavy rains immediately preceded the initial failure. Ground water was and is present in large quantities. Additional water was and may still be entering the fill from buried coal mines.

6. Description of rock or soil types involved in the slide and in the adjacent area

The cut to fill and fill to cut sections are on shales and sandstones of the Kootonai Fm. The fill foundation is composed of soft clay to a depth of 30' to 40', hard clay and gravel to depths of 40' to 60' and shale bedrock at depths of 50' to 60'.



Belt Slide
Note - water ponded on upstream side of fill, also old road which is approximate elev. of old coal mine.
Plate No. 1

Belt Slide

7. Investigations: A preliminary investigation of this alignment was performed by State Highway Personnel together with members of the Unites States Geological Survey, Engineering Geology Branch. The main purpose of this study was to classify the materials to be excavated in the large cut section immediately west of the large fill which failed. In this investigation the geological formations were mapped by both surface and subsurface methods. The fill foundation, which failed, was given cursory appraisal but, inasmuch as it contained a great deal of granular material, the consensus of opinion was that it would rapidly consolidate and gain strength under the fill weight and would, therefore, present no problem. (1)

(B) Following the failure in 1964 further investigation was undertaken by State Highway Personnel. This study included drilling five test holes below the fill to obtain samples of the foundation materials. From these test holes, samples selected were tested for shear values, density, natural moisture and all normal soils tests. In all, 29 samples were tested for shear strength by unconfined shear testing. Shear strengths obtained by this testing ranged in value between 0.3 to 3.5 tons per square foot, averaging 1.2 tons per square foot. (Triaxial shear tests indicated a much higher shear strength.) The shear plane was determined to approximate a circular arc and, therefore, this configuration was used in the stability analysis. Graphic reconstruction of the fill and foundation was undertaken to determine the shear values at the time of failure. For this analysis the portion of the arc through the fill was considered to have a shear resistance of a 35° angle of internal friction with no cohesion. This value was assumed because the fill contained sandstone excavated from the cut section just west of the fill. Using a safety factor of 1.00 at the time of failure, the foundation shear strength was determined to approximate 800+ pounds per square foot. Utilizing these shear values, a counter balance was designed to raise the safety factor to about 1.30±.

(1) Various Montana Highway Commission Memoranda - dated 1953.

7. Investigations, Con't.

In addition, the sub-surface materials near the fill toe were found to be saturated, water was found to be entering the fill from a coal adit covered by the fill and a very high ground water table was discovered.

Curative measures proposed at this time vere as follows:

(1) Construction of a counterbalance together with fill slope flattening.

(2) Sub-excavation at the base of the fill to be backfilled with granular material to provide drainage in this area.

(3) Perforated pipe drains to the buried coal mines, and at the toe of the fill parallel to centerline.

These curative measures were undertaken in 1965 and were virtually complete when a series of compressional cracks were observed on the counterbalance indicating an excessive strain and a near-failure condition.

(C) The counterbalance was in place, the drains established and the fill slope flattening completed to approximately 40 feet below the finished grade line when the compression cracks developed on the berm of the newly completed fill slope. Further placement of fill materials was stopped and additional counterbalance was added at this time. This action stopped the sliding with movement attributed to weight adjustment, indicating settlement of the fill addition and the berm.

Further analysis of this slide indicated that the shear strengths used in the original analysis were probably too high and that with the lower shear values the enlargement of the counterbalance would probably not in itself suffice as a curative measure, (within economic reason at least).

7. Investigations, Con't.

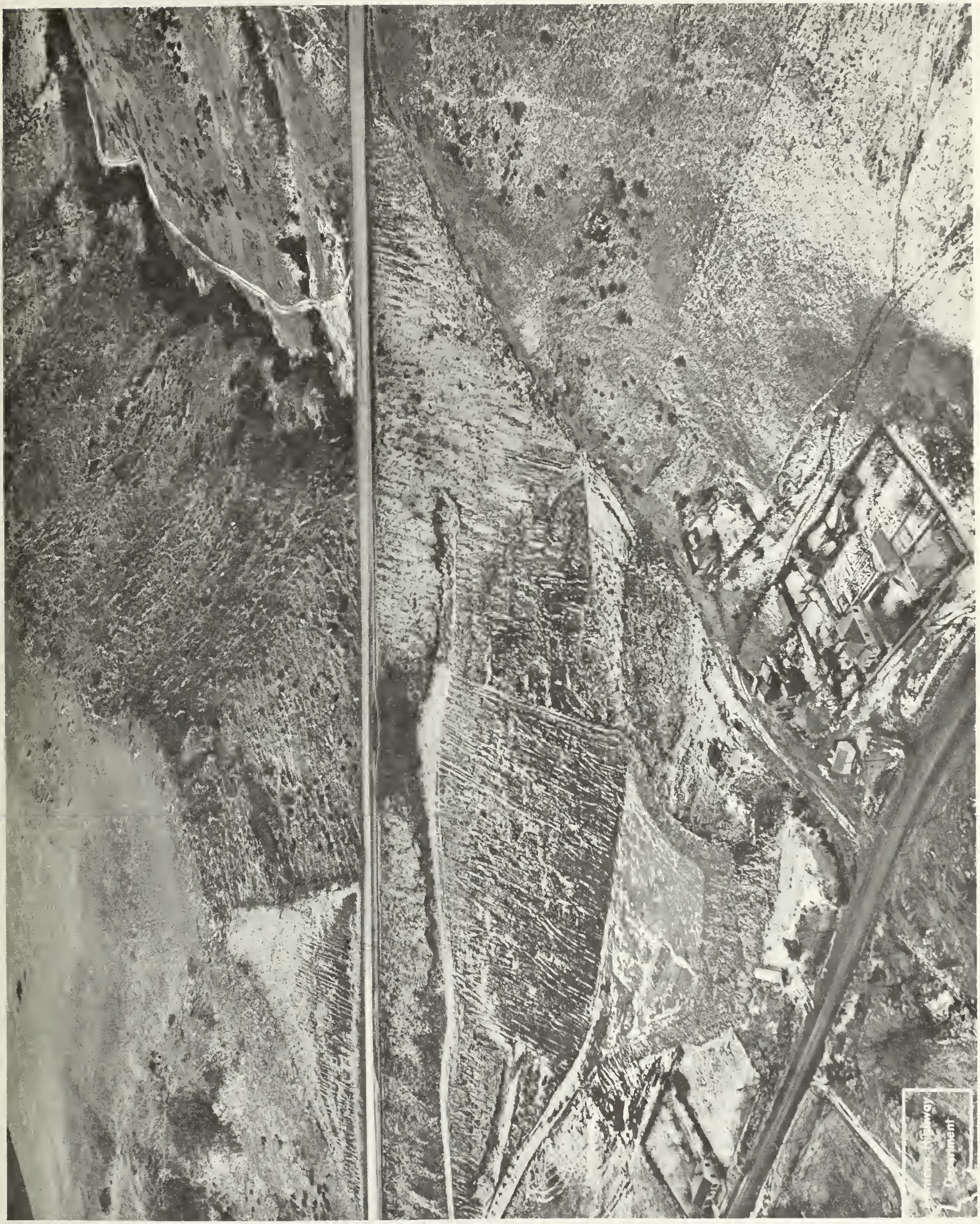
The additional investigation for the above unstable condition was carried forth by State Highway personnel and included drilling test holes both below and above the slide. During this study, one test below the fill on the downstream (north) side, encountered a very permeable gravel layer in an old erosional channel cut into the shale bedrock. Geological investigation indicated the presence of a permeable sandstone beneath the shale so a test hole was completed to find and test this sandstone with the idea that perhaps if dry and permeable, vertical drainage of the fill foundation might be accomplished. This solution failed because the sandstone contains an artesian flow, and thus would compound the problem if a passage was opened to the foundation materials. To better evaluate the ground water situation, drilling was initiated above the fill and this proved that a large flow of water was involved. A 50 hour continuous pumping test indicated that removal of 48 gallons per minute from the aquifer above the slide, lowered the water table below the slide by 0.7± feet. The artesion flow, previously mentioned, stopped and it was thus determined that dewatering was possible.

This investigation also disclosed the possible existance of another abandoned coal mine, believed to be very wet, which could be supplying the fill with water from the canyon wall.

Shear strengths of the foundation materials were obtained by the use of Triaxial shear apparatus and indicated the following values:

In the siltier portions - Ø = 22°, Cohesion = 575 lbs/ft.^{2}

In the clayey portions - Ø = 8°, Cohesion = 1150 lbs/ft.^{2}



Belt Slide

Note - downstream side of fill showing condition of reconstruction at the time of excessive strain which resulted in a redesign of the corrective measures.

Plate No. 3

Belt Slide

8. Causes: It is obvious that the heavy rainfall recorded in 1964 caused a slide to develop in a fill section which had apparently been stable for 10 years. It is also obvious that water is still available to continue this same unstable condition. What has caused this condition? Apparently some combination of the following factors are responsible.

Geologic and Physiographic setting.

The valley walls at Frenchmans Creek which is the setting of the failure area in question, are carved in sandstone and shales of the Kootenai formation of Cretaceous age. The sandstones commonly are aquifers, and the interbedded shale layers confine this water and promote some hydrostatic head.

Underlying the Kootenai formation and underlying the fill section at the fill bottom is the Morrison formation of Jurassic age. This unit is composed in part of extremely water sensitive marine clay shale.

Structurally, the formations involved dip gently toward the north in the area and underly a pediment gravel surface which nearly parallels the bedding attitude for some miles. The sandstones in the Kootenai formation for many square miles up structure from the failure area lie at or near the surface and thus have a tremendous recharge area for ground water entry. The water becomes confined by shale layers and is forced under hydrostatic head into the failure area. The stream eroded canyons such as the Frenchmans Creek canyon cut into these sandstones and water is thus available to saturate the area. Additional water is supplied by surface runoff into the surface drainage features. The very heavy rainfall of 1964 contributed to both these water supplies and no doubt the affects of the above normal rainfall for the past several years will affect the groundwater for some time to come. Placement of the fill compressed

8. Causes, Con't.

the foundation materials, creating hydrostatic pressures on the gravel aquifer under the fill. These pressures forced water into the clay and silt above and probably into the fill itself.

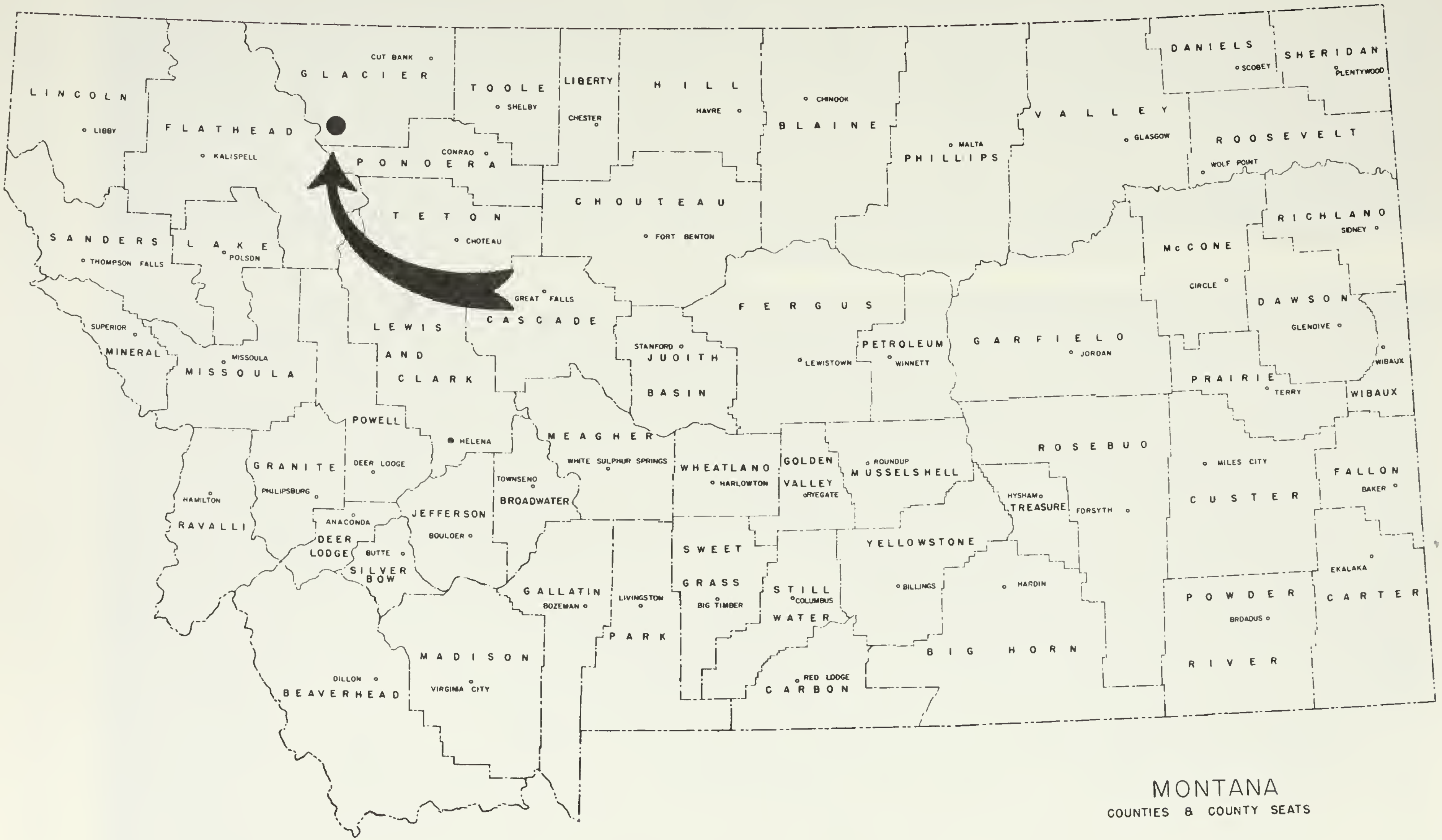
Old coal mines which gather water from the afore mentioned sources have no doubt contributed additional water to the fill.

9. Possible Corrective Measures: Inasmuch as the major reason for failure is foundation saturation of the clay and silty clay layers present, the main solution apparent at this time is dewatering. Because the major source of water is the tremendous underground flow; dewatering the aquifer above the fill seems to be the best solution. A recommendation to establish pumping facilities above the fill toe and attempt to dewater the gravel aquifer to a depth of at least 40 feet has been submitted. This will relieve the hydrostatic pressure under the fill and allow the relief of pore pressures into this aquifer. This should cause drying to take place in the foundation materials and in the fill materials with an expected gain in shear strength.

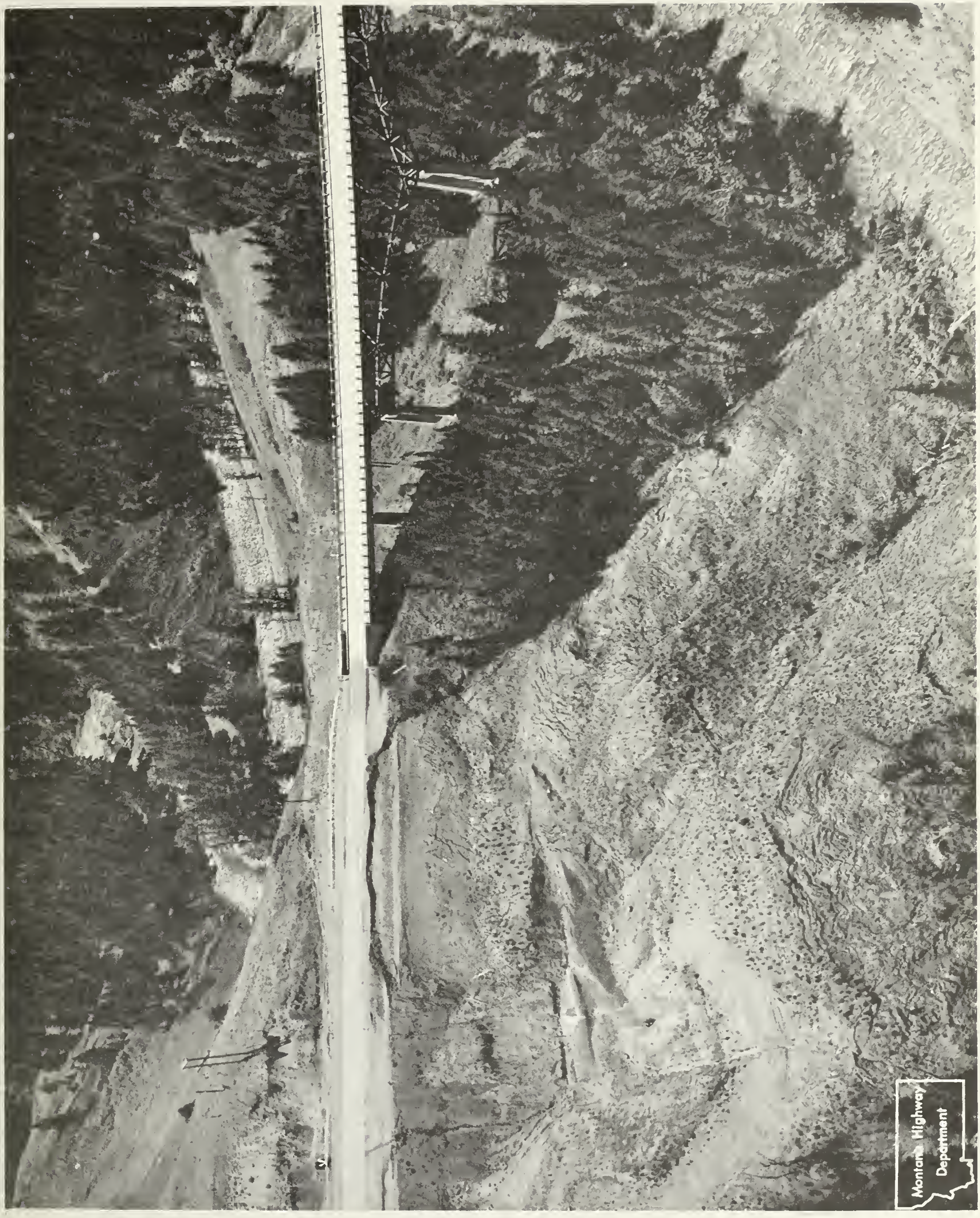
Additional berm or counterbalance has been proposed to safe guard the fill while dewatering takes place and several additional test holes have been proposed. The test holes are for the purpose of locating possible sources of water entering the fill from the walls of the canyon, whether these be old coal mines or sandstone layers. It is that these can be adequately drained by horizontal holes if and when discovered.

10. Data for Avoidance of similar conditions in the future: Although preliminary studies were conducted by competent personnel, this trouble area was not recognized as such. The clue here, that apparently was overlooked, is that throughout this area, where the Kootenai formation is exposed over the Morrison formation, natural landslides have developed. The cause is apparent because the soft sensitive shales in the Morrison formation became saturated by water in the sandstone and coal seams in the Kootenai formation and wherever disturbed by loading or undercutting a failure occurs.

EAST GLACIER LANDSLIDE



SLIDE NUMBER A-3



East Glacier Slide
Note - slide development, this picture depicts one of the many movements. Some of which were more extensive.
Plate No. 1

Landslide No. A-3
East Glacier Slide

LANDSLIDE EVALUATION FORM

1. Location County Glacier Township 31N Range 12W Sec. 17

Highway U. S. 2 1 Mile east of East Glacier

2. Dates of study 1945 to 1962

Remarks concerning general weather and rainfall/snowfall conditions during the time of study.

Area of moderate precipitation

3. Date of origin 1945 Date Movement Ceased 1962 ?

Most active season Unknown

Relative activity (speed, etc) from origin to present 1945 to 1962, slow

Movement related to spring runoff, etc. (See plate No. 1)

4. Physical dimensions Length 500 feet, Width 360 feet maximum

Approximate yardage involved (Estimated @ 20,000)

Height of cut/fill slope Variable

Angle of cut/fill slope 1½ to 1 fill slope

5. Water table, rainfall information, and remarks concerning the effect of water in the unstable condition High ground water table with the sub-surface drainage toward the river following thin sandstone beds or the top of the shale. Construction cut off these natural drainages, which caused the build up of pore pressures in the overlying clay and silt.

6. Description of rock or soil types involved in the slide and in the adjacent area

Some stratified glacial till over Colorado shale of Cretaceous age. The shale strikes @ S70°E and dips 15° to 25° NE. The shale contains thin lenses of sandstone and is badly fractured with numerous faults cross-cutting the bedding.

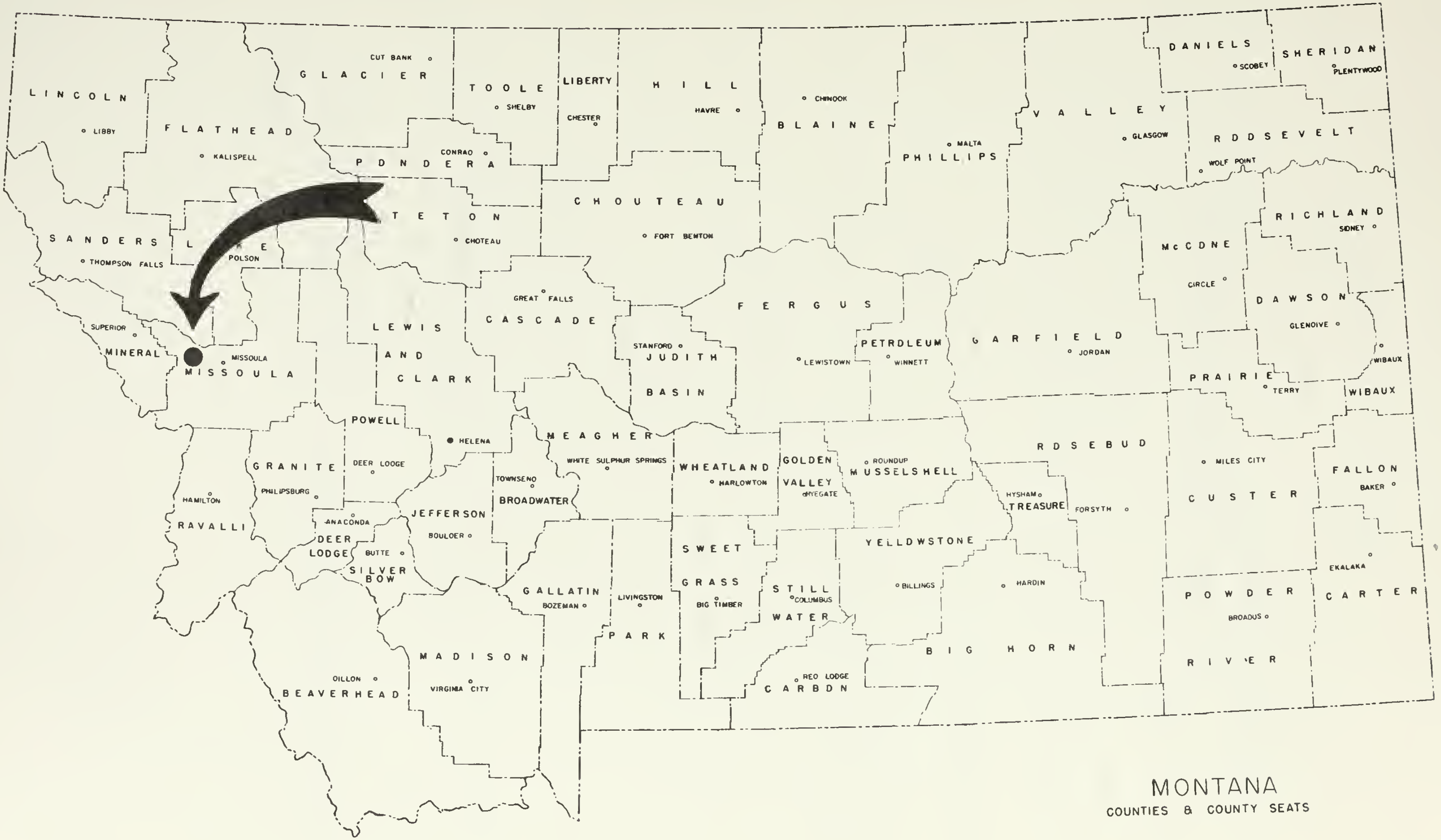
7. Investigation and Causes: Investigation of this slide by maintenance personnel prior to 1959, resulted in periodic additions of material to bring the roadway back to grade. A rebuilding and slight line change was undertaken in 1958. These measures did not correct the situation. In 1959, a more extensive geological investigation was undertaken which included both horizontal and vertical drilling. This investigation found and partly delineated the slide plane, the water table elevation and the elevations of sound bedrock. Horizontal holes were drilled in an attempt to drain the sliding area but met only indifferent success as the shale was relatively impermeable and would not drain into the small diameter drill holes in sufficient quantity to cure the sliding condition. The fill, composed of glacially derived silts and clays, was placed over a fissil black shale. This shale outcrops on a steeply sloping surface toward the Two Medicine River which has cut a deep 200 foot ± gorge in the relatively soft shales. Bedding in the shale also slopes toward the river. The relatively flat ground surface into which the river has cut its canyon reflects its glacial origin. In the area south and east of the slide, away from the river, very poor surface drainage exists which creates a constant recharge area for ground water. This is attested to by numerous springs and seeps which occur in the general slide area. The original fill placement evidently cut off part of this natural ground water drainage, thus providing seepage pressures and loss of shearing strength which initiated a failure.

8. Possible Corrective Measures: Subsurface drainage by placement of horizontal drains did not adequately drain the saturated clay shale. The sliding condition was finally "cured" by a complete removal of all materials involved and a complete rebuilding of the fill structure. It now appears to be stable.

East Glacier Slide

9. Data for avoiding similar situations in the future: The presence of pothole lakes and lack of surface drainage is an indication that ground water problems can be expected. This situation together with moisture sensitive foundation materials such as glacial till and clay shale usually indicates that sliding conditions can easily develop, especially when placing a fill over a sloping surface. Adequate foundation preparation, including benching, drain blankets and perforated underdrains may be necessary when dealing with this type of problem.

FRENCHTOWN LANDSLIDE



MONTANA
COUNTIES & COUNTY SEATS

SLIDE NUMBER A-4

Landslide No. A-4

Frenchtown Slide

LANDSLIDE EVALUATION FORM

1. Location County Missoula Township 14N Range 21W Sec. 1

Highway U. S. 10 Project No. ± 2 Miles East of Frenchtown

2. Dates of study July, 1961, July-August 1964, April 1965

Remarks concerning general weather and rainfall/snowfall conditions during the time of study.

This failure does not seem to respond directly to rainfall, however, excessive rainfall no doubt contributes indirectly to the problem.

3. Date of origin pre-highway Date Movement Ceased Still unstable

Most active season Unkown

Relative activity (speed, etc) from origin to present Intermittent periods of fairly rapid movement, followed by long periods of little or no movement.

4. Physical dimensions 300 ± feet X 200 ± feet in lateral extent.

Approximate yardage involved 70,000 ± cubic yards

Height of cut/slope 30 ± feet

Angle of cut/slope Original cut slope unknown

5. Water table, rainfall information, and remarks concerning the effect of water in the unstable condition

The water table lies at or just above the contact of Pleistocene lake deposits and the underlying cemented Tertiary gravel.

6. Description of rock or soil types involved in the slide and in the adjacent area

Glacial Lake Missoula sediments of varved silt and clay over cemented Tertiary gravel beds. The gravel beds dip toward the south (toward the highway).

Frenchtown Slide

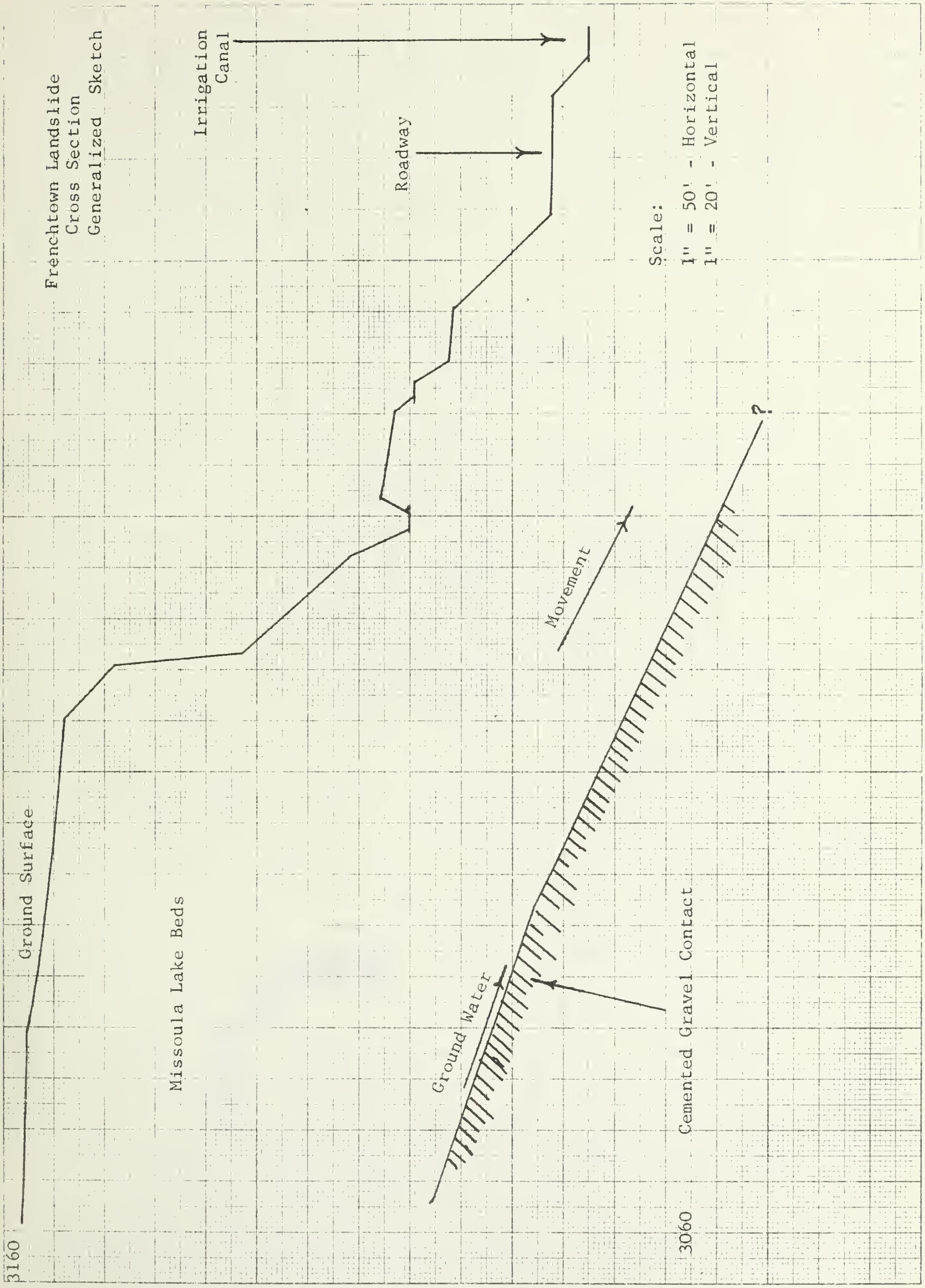
7. Investigation and Cause: The original geological investigation in 1961 discovered the ground water problem but did not accurately determine the slide plane or the ground water source. Additional work in 1964 defined the ground water table and correctly identified the underlying cemented gravel layer and slide plane. This investigation included the drilling of nine test holes, correlating between test holes with a portable refraction seismograph and tracing the ground water flow.

   Water enters the ground at the base of the mountains to the north and follows the top of the cemented gravel providing lubrication to the clay and silt. The blockglide movement followed this lubricated surface downhill toward the roadway, with movements restricted somewhat by the irregular gravel surface. This irregular surface provided frictional resistance to movement which was overcome when the clay became sufficiently saturated to lower the cohesion to a value that allowed movement to take place. After some movement, the resistance of the gravel-silty clay contact evidently held until the process repeated itself again.

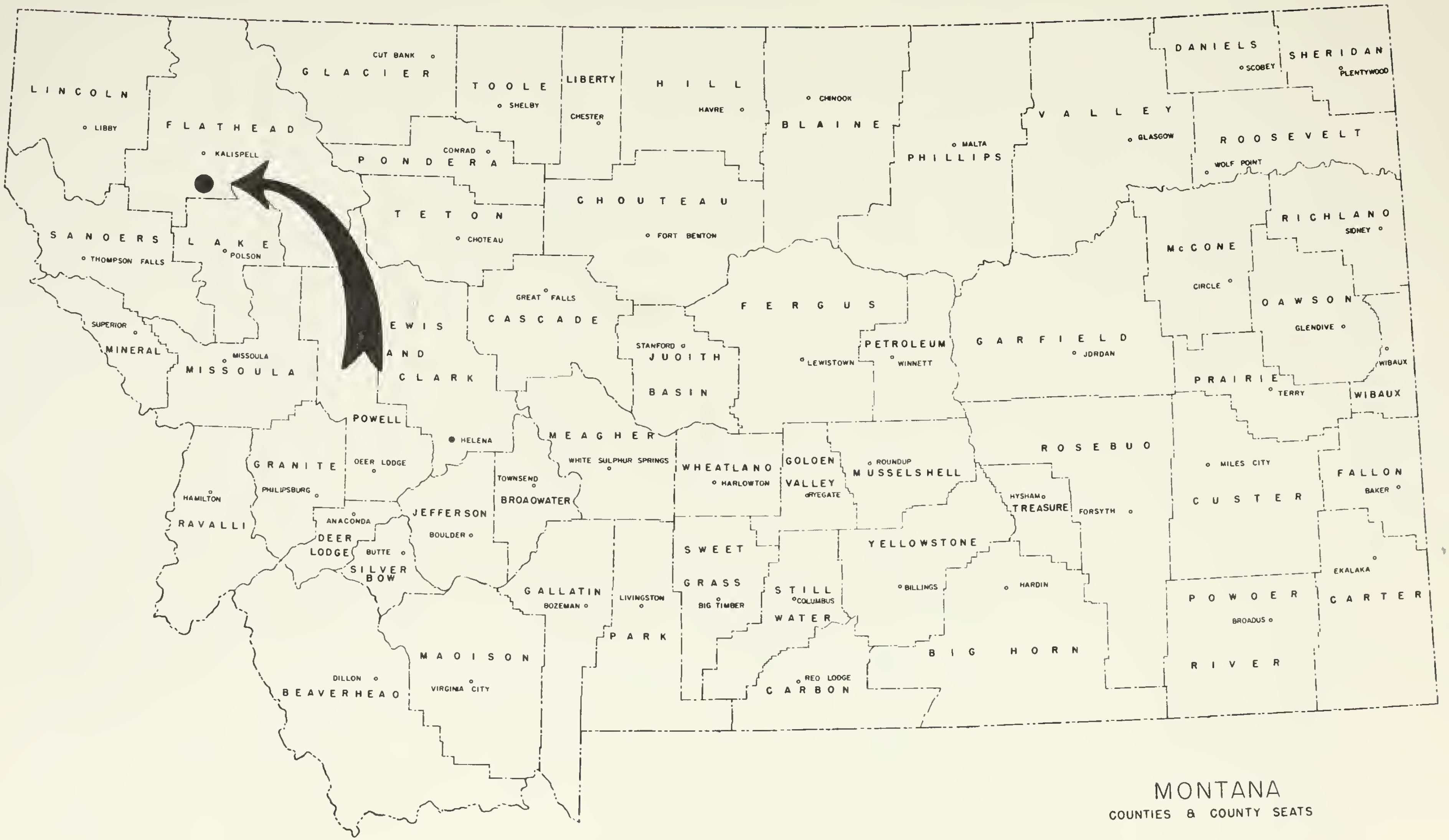
8. Corrective Measures: Partial correction was accomplished when a great deal of the sliding material was removed from above the highway and placed as a counterbalance wherever possible. Interstate highway routing in the area will be on or near this slide. We have recommended either a line change to the south to completely avoid this slide or complete removal of the lake bed materials, leaving the highway foundation on the gravel layer.

Frenchtown Slide

9. Data for avoiding similar conditions in the future: Glacial Lake beds are prevalent on the mountain valleys of Western Montana. They are easily recognizable as they have a distinctive pink color and a varved (thin-bedded) structure. The clays are usually not highly plastic but are sensitive to changes in moisture. Wherever a high fill or deep cut is planned, a careful investigation of ground water conditions is necessary. In addition, this varved clay and silt becomes unstable in cut slopes because natural drainage is disturbed. Cut slopes in this material should be as steep as the shear resistance will allow. Inasmuch as shear resistance is dependent upon moisture content; detailed preliminary study is necessary to determine the natural moisture and subsurface drainage conditions.


Frenchtown Landslide
Cross Section
Generalized Sketch
Irrigation Canal
Roadway
Scale:
1" = 50' - Horizontal
1" = 20' - Vertical
Ground Surface
Missoula Lake Beds
Movement
Ground Water
Cemented Gravel Contact
?
3160
3060

FLATHEAD LANDSLIDE



MONTANA
COUNTIES & COUNTY SEATS

SLIDE NUMBER A-5

Landslide No. A-5

Flathead Slide

LANDSLIDE EVALUATION FORM

1. Location County Flathead Township 26N Range 20W Sec. 18

½ Mile South of Lakeside

Highway U. S. 93 Project No. F 191(24) Station 1747± to 1772±

2. Dates of study April & May 1964

Remarks concerning general weather and rainfall/snowfall conditions during the time of study.

Not pertinent

3. Date of origin 1964 Date Movement Ceased 1964

Most active season 1964

Relative activity (speed, etc) from origin to present the rockfall portion occurred rapidly.

4. Physical dimensions Random rockfalls and debris slides

Approximate yardage involved Unknown

Height of cut/slope 70 - 80 feet

Proposed slope ½ to 1

5. Water table, rainfall information, and remarks concerning the effect of water in the unstable condition not pertinent

6. Description of rock or soil types involved in the slide and in the adjacent area

Calcareous argillite in the center of the cut section with glacial till covering the argillite in various depths.

Flathead Slide

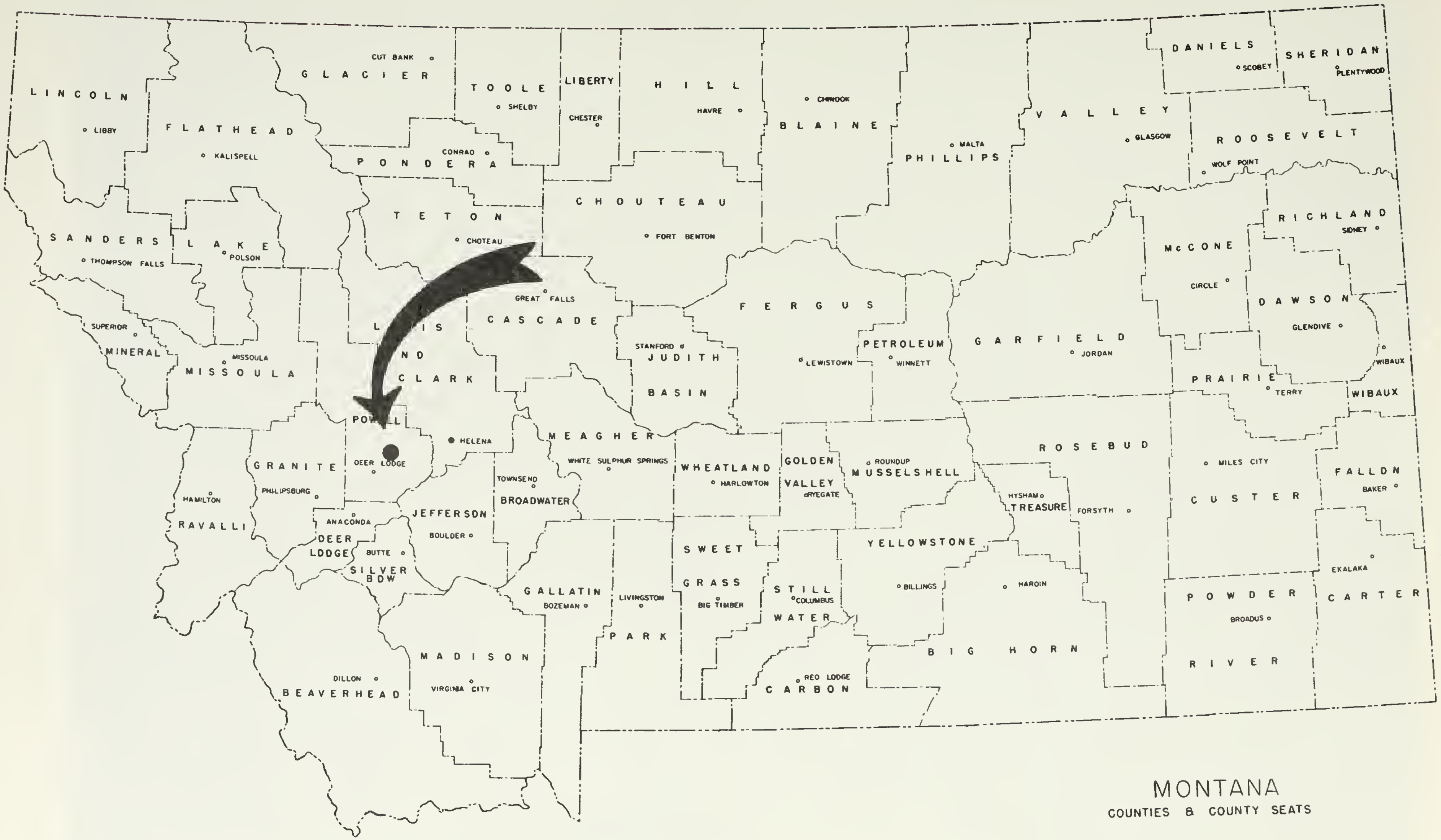
7. Investigation and Cause: During a geological investigation it was determined that the hill through which the cut was made contained a core of calcareous argillite. This rock contained two major planes of weakness; these being the bedding plane which dipped at 33° toward the southeast, and a major joint pattern which dipped at 70° southwest. Because the highway trends roughly north-south through this cut section, it was apparent that the west side was very unstable because of the existing bedding and joint patterns.

   In addition the rock was overlain by glacial till which increased in thickness both north and south. The glacial till, which has an engineering classification of A-4(3), obviously was not stable on the previously recommended steep cut slope. As a result the rock was failing in large blocks from the west side because of the bedding and joint patterns while the till was failing because of the lack of internal strength.

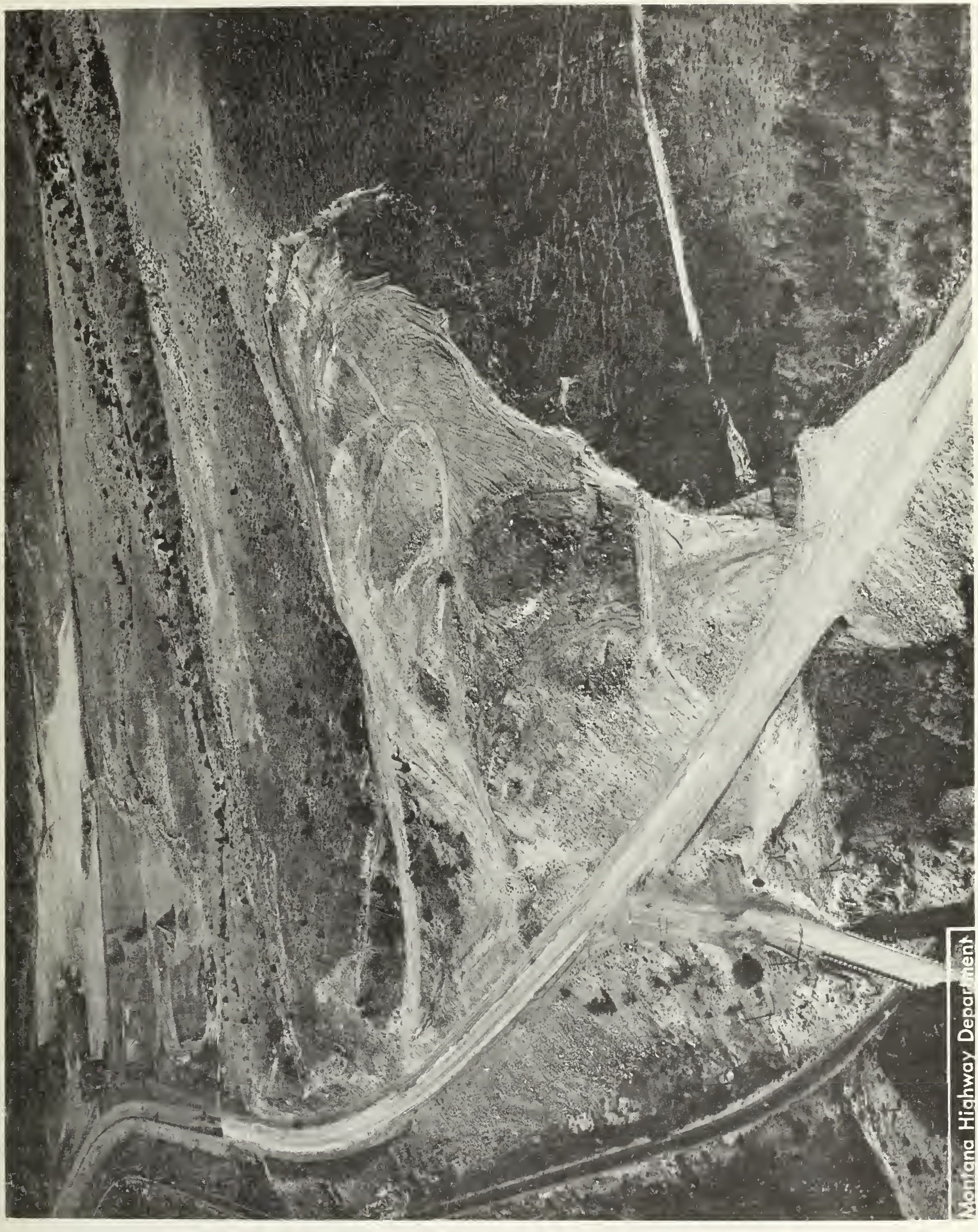
8. Curative Measures: The west cut slope, where in rock, was flattened to conform better with the structural stability of the rock and the till was flattened to better conform with its internal strength.

9. Data for avoidance of similar conditions in the future: Deep cut sections such as this one should be explored at least to the depth of the proposed grade line in the preliminary, planning stage. Data obtained during this preliminary investigation can then be utilized to properly design cut slopes.

GARRISON-AVON LANDSLIDE



SLIDE NUMBER A-6



View looking northwest of the slide during the construction of the original corrective measure of head removal.

Landslide No. A-6

Garrison-Avon Slide

LANDSLIDE EVALUATION FORM

1. Location County Powell Township 9N Range 9W Sec. 11

Highway U. S. 12 Project No. F-249(11) Station 248± to 290±

2. Dates of study 1958, Sept. 1960, July 1963, Oct. 1963

Remarks concerning general weather and rainfall/snowfall conditions during the time of study.

The slide mass was saturated by rain and/or snow intermittently with resulting sporadic and rapid movements.

3. Date of origin Winter 1958 Date Movement Ceased 1963

Most active season June 29 & 30, 1963

Relative activity (speed, etc) from origin to present Rapid movements followed by subsequent quiescent periods (the toe moved 78 feet on June 28 & 30, 1963).

4. Physical dimensions 700' X 700', with slide block 0 to 40 ft in depth.

Approximate yardage involved 280,000 ± cubic yards.

Height of cut/fill slope 70' ±

Angle of cut/fill slope 1:1 (45°)

5. Water table, rainfall information, and remarks concerning the effect of water in the unstable condition The water table followed the sandstone-shale contact and saturated the overlying clay shale.

6. Description of rock or soil types involved in the slide and in the adjacent area

"The material involved in these slides is the Colorado shale formation composed of light gray sandstone, gray-green silty shale, and black shale; at least two distinct sills of diorite 10 to 15 feet thick have intruded the part of the formation involved in the slide. The sandstone is medium hard and fairly durable, the hard, massive diorite alters only slowly;

Garrison-Avon Slide

6. Con't.

the black shale is fissile and, although it disintegrates when exposed to the weather, it does not alter rapidly; the massive gray-green shale slakes and alters readily to clay. The beds strike about N. 60°E. and dip 10 to 20°S.E. The slope of the ridge top is about 15° to the southeast. At least two sizeable faults are exposed in the cut and both of these figure prominently in the movement; the faulting and the sill intrusion intensively fractured the formation contributing appreciably to alteration and overall weakening of the rocks.

The east slide is moving in a trough formed by the intersection of two faults. The trough pitches to the south (into the cut) and the sliding material is badly shattered and altered. As the faults diverge to the south the material exposed between them is much less fractured which doubtless contributed to stability before the cut was made. This slide constitutes only a small part of the total moving material, but it has consistently been the most active part.

Most of the sliding material is in the west slide which is potentially more dangerous and will certainly be more difficult to cure. This slide is moving directly into the cut along one or several very altered beds of gray-green shale about 45 feet higher than the roadway; above these beds the excavation has exposed numerous zones of shattered rock and mud which are also obviously involved in the movement. The slide plane may be either reasonably straight in the direction of movement or concave upwards. With a straight slide plane (a block-type slide) removal of material from the head portion would have much less effect on movement; that is, any major portion of the slide unexcavated would still be on a plane too steep for stability. If the slide plane is considerably curved (a rotational-type slide), removal of material from the head should retard movement. As far as could be told from this brief examination, the slide is much closer to being the block type (see photo, page 43),

6. Con't.

but detailed measurements over a period of time would be required to substantiate this. Certainly, no toe area, seriously impending movement, is now present.

In either case the sequence of events causing movement follows this sequence: the rock, which has been extensively broken by faulting and sill intrusion, begins to alter and disintegrate; rain water percolates readily into the fractured rock carrying along the clayey silty alteration products until the altered shale beds prevent further penetration; the combination of added weight in the total mass and increased moisture in the critical shale beds results in movement which continues until some obstruction is encountered, or the friction along the slide plane increases, or the balance is changed."(1)

7. Investigation and Cause: The slide was investigated by Project and Division personnel during periods of failure during the original cut construction. The first formal investigation occurred in Sept. 1960 and the slide description above was part of the Sept. 24, 1960 report. (See plates 1 and 2, pages 48 and 49).

Following the attempt to remove a portion of the landslide "head" in 1960-1961; surveyed control lines were placed and subsequently checked for movement at regular intervals. Very little movement occurred until 1963 when the slide reactivated and rapidly moved 78 feet, closing the highway.

Investigation following this movement included accurate definition of the slide plane by drilling and refraction seismic survey and determination of the possible curative measures.

(1) Charles Van Alstine, Inter-Department Memorandum Sept. 24, 1960

Garrison-Avon Slide

8. Curative Measures: The original attempt to remove weight from the slide "head" failed because this was a block glide slide. The saturated clay on the slide plane had little or no internal friction and as the water content increased, the cohesion forces approached zero. The actual slide plane along which movement took place dipped 11° toward the cut section.

The only possible solution was either to move the highway away from the slide or to remove a major portion of the moving mass. The latter solution was undertaken in 1963 and completed in 1964. This landslide is now considered cured.

9. Data for avoiding similar situations in the future: Detailed preliminary geologic mapping, combined with a knowledge of the principals of soil mechanics could have indicated the possibility of slide development. The bedding dips and the presence of faulting plus the existence of a soft clay shale over a potential slide surface combined to promote sliding activity. Such preliminary mapping is necessary where soft shales are included in the geologic section. These Cretaceous age sediments are particularly condusive to slide development.

GARRISON - AVON LANDSLIDE
PLAN VIEW

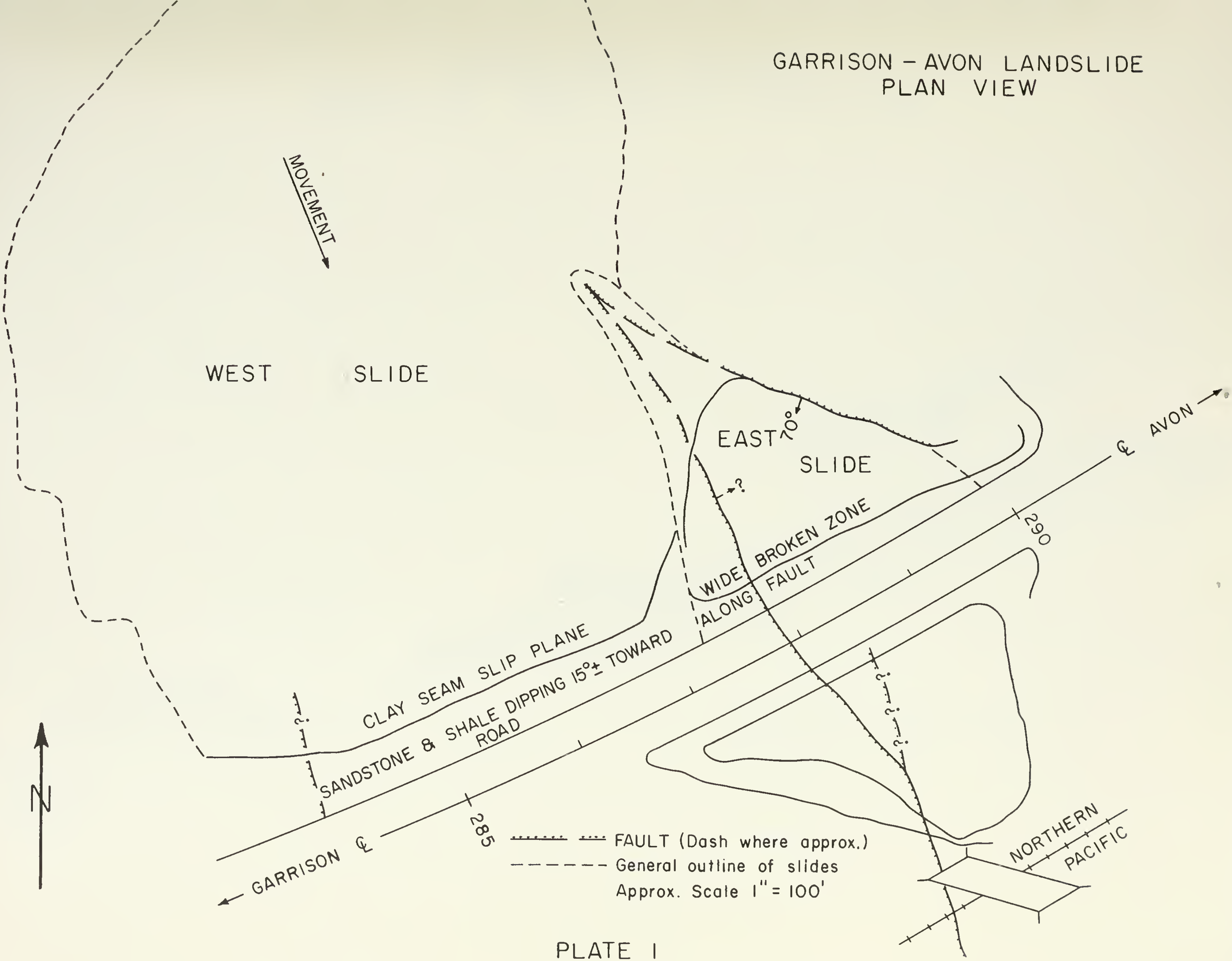

MOVEMENT
WEST SLIDE
EAST SLIDE
10°
?
WIDE BROKEN ZONE ALONG FAULT
CLAY SEAM SLIP PLANE
SANDSTONE & SHALE DIPPING 15° ± TOWARD ROAD
℄ AVON
290
285
GARRISON ℄
NORTHERN PACIFIC
N
FAULT (Dash where approx.)
General outline of slides
Approx. Scale 1" = 100'

PLATE I

GARRISON-AVON LANDSLIDE
GENERALIZED CROSS SECTION LOOKING WEST

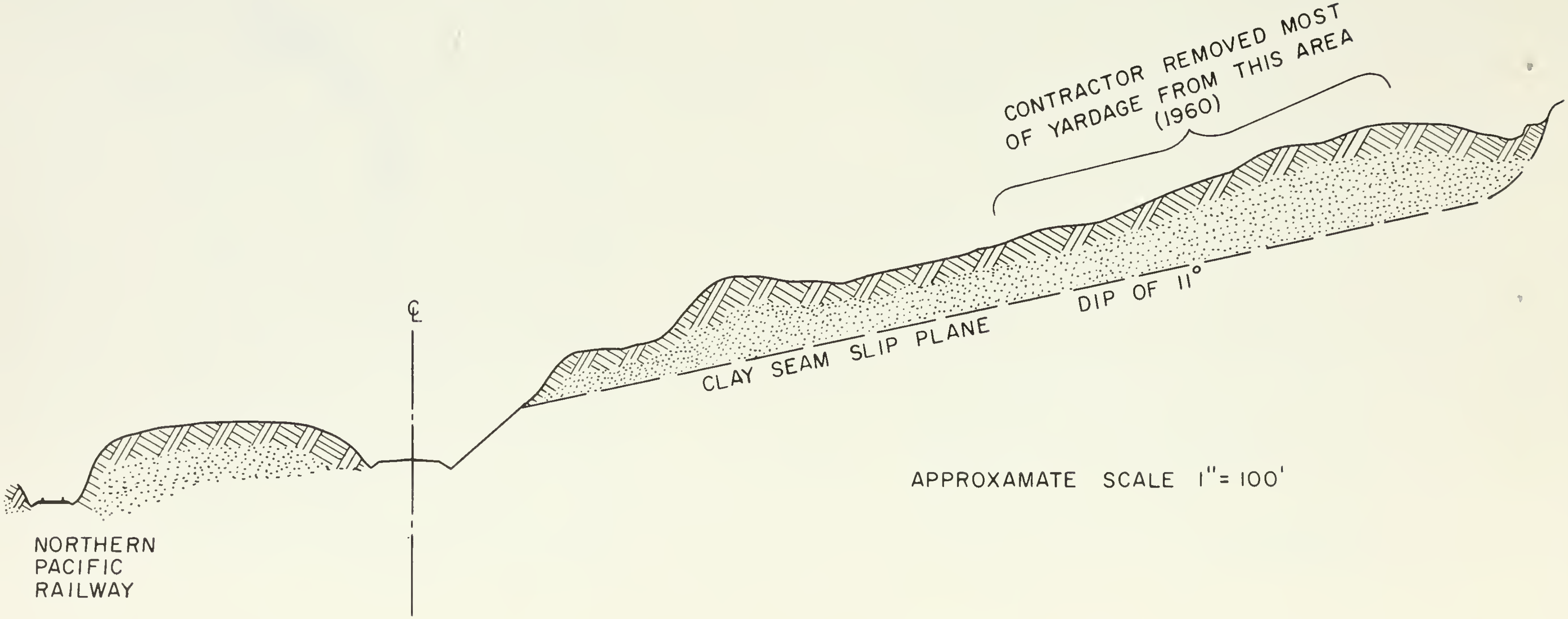
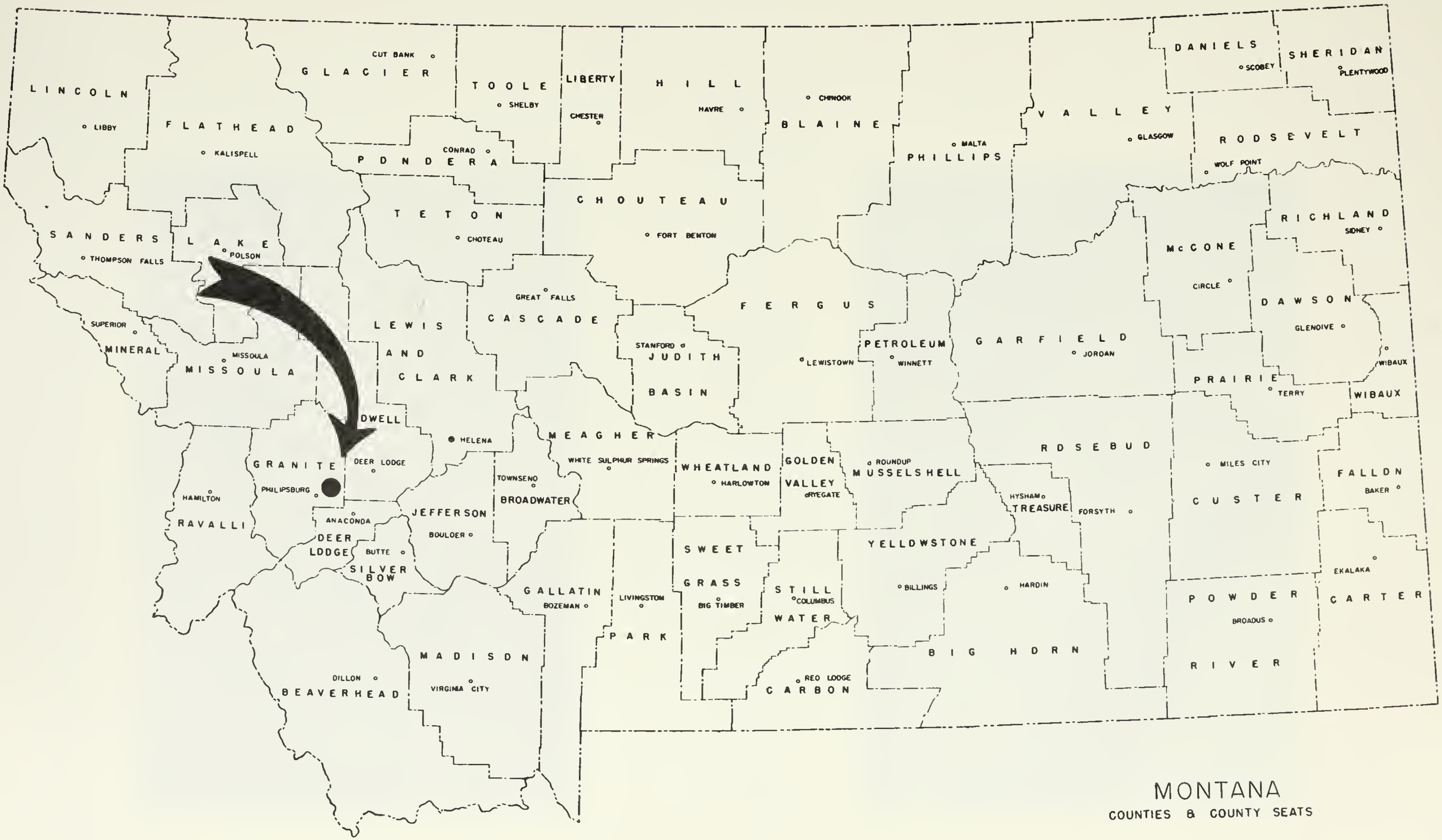


PLATE 2

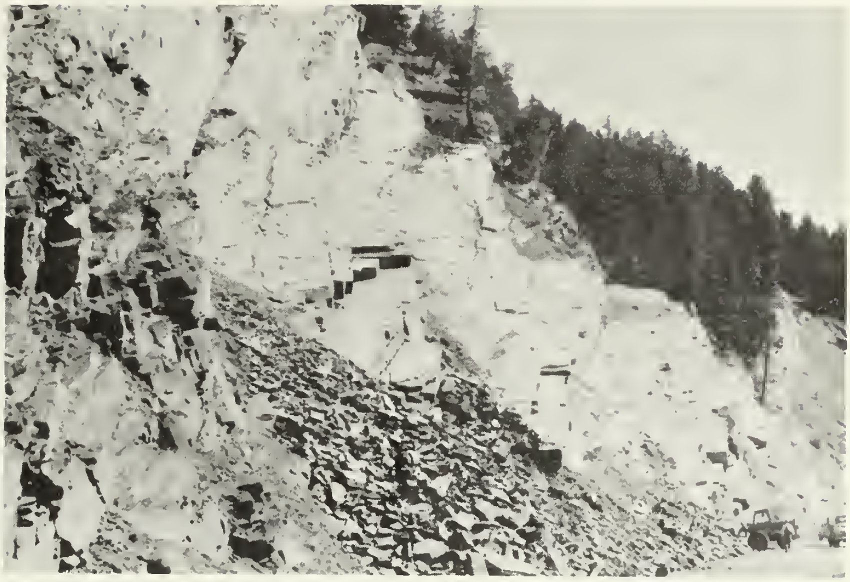
GEORGETOWN LAKE LANDSLIDE



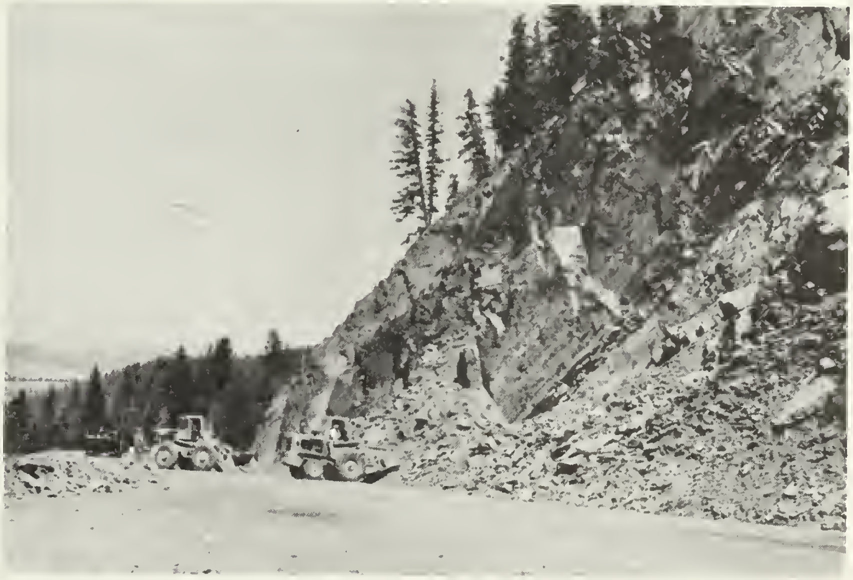
SLIDE NUMBER A-7

Plate No. 1

Georgetown Lake Slide



The bedding planes face the camera in this picture. Note steep component toward the roadway. Two distinct joint patterns can be seen near the center of the picture.



In the picture, the bedding dip is toward the lower left. One major joint pattern nearly parallels the highway and dips vertically.

Landslide No. A-7

Georgetown Lake Slide

LANDSLIDE EVALUATION FORM

1. Location County Granite Township 5N & 6N Range 14W 14W Sec. Sec. 1 Sec. 36

Highway U. S. 10A ± 8 miles South of Philipsburg

2. Dates of study Sept. 1964

Remarks concerning general weather and rainfall/snowfall conditions during the time of study.

Heavy rainfall or melting snow lubricates the planes of weakness and promotes sliding.

3. Date of origin Unknown Date Movement Ceased Still active

Most active season

Relative activity (speed, etc) from origin to present Very rapid movement during failures.

4. Physical dimensions 2600 feet along the highway

Approximate yardage involved

Angle of cut/slope Originally cut very steep.

5. Water table, rainfall information, and remarks concerning the effect of water in the unstable condition Rainfall or melting snow lubricates the rock and allows blocks created by joints and frost activity to slide onto the highway.

6. Description of rock or soil types involved in the slide and in the adjacent area

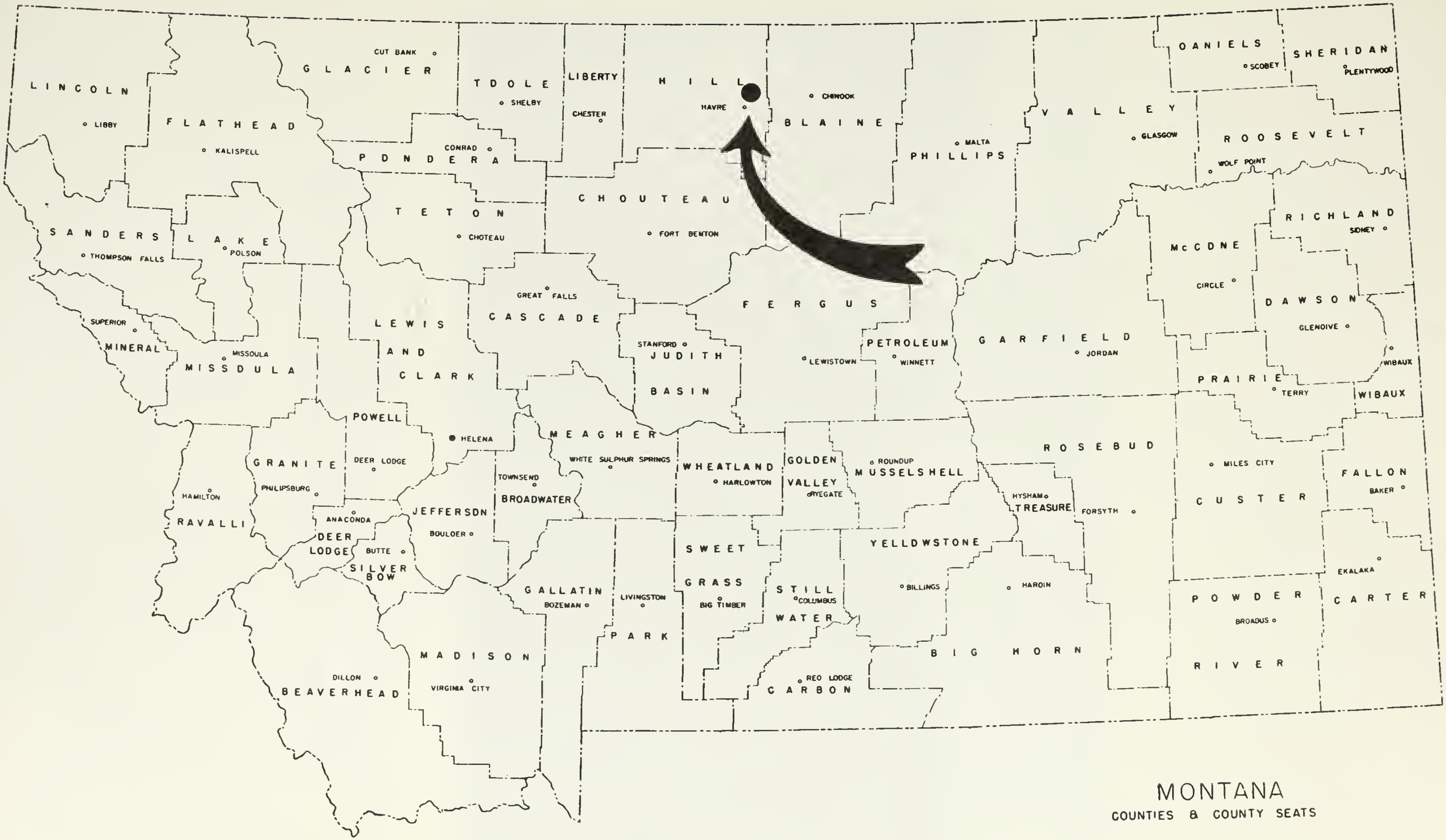
Massive quartzite and thin bedded argillite with bedding dips toward the highway. Several pronounced joint patterns adversly affect the cut slope stability. See Plate No. 1, page 51.

Georgetown Lake Slide

7. Investigation and Cause: Detailed geologic mapping was conducted in Sept. 1964 of this section of highway. Original highway construction cut the rocks as steeply as possible in the cut slopes on this winding road which follows a steep rock mountain side. Where the highway nearly parallels either the bedding or major joints or one of several faults, rock slides have developed. Several closely spaced joint patterns which dip steeply away from the highway at various angles cause large blocks of the rock to break away from the main rock mass and slide onto the highway. The slide plane in each case is the bedding surface which dips at 38' to 56° toward the highway, or a fault zone which dips at 38° toward the highway. During periods of heavy rainfall, the bedding surfaces become wet and lubricated allowing the previously loosened blocks to slide onto the travelway.

8. Data for avoiding similar situations in the future: Preliminary geological study of the rocks involved in highway cuts will quickly determine the proper cut slope design to provide maximum cut slope stability.

HAVRE LANDSLIDE

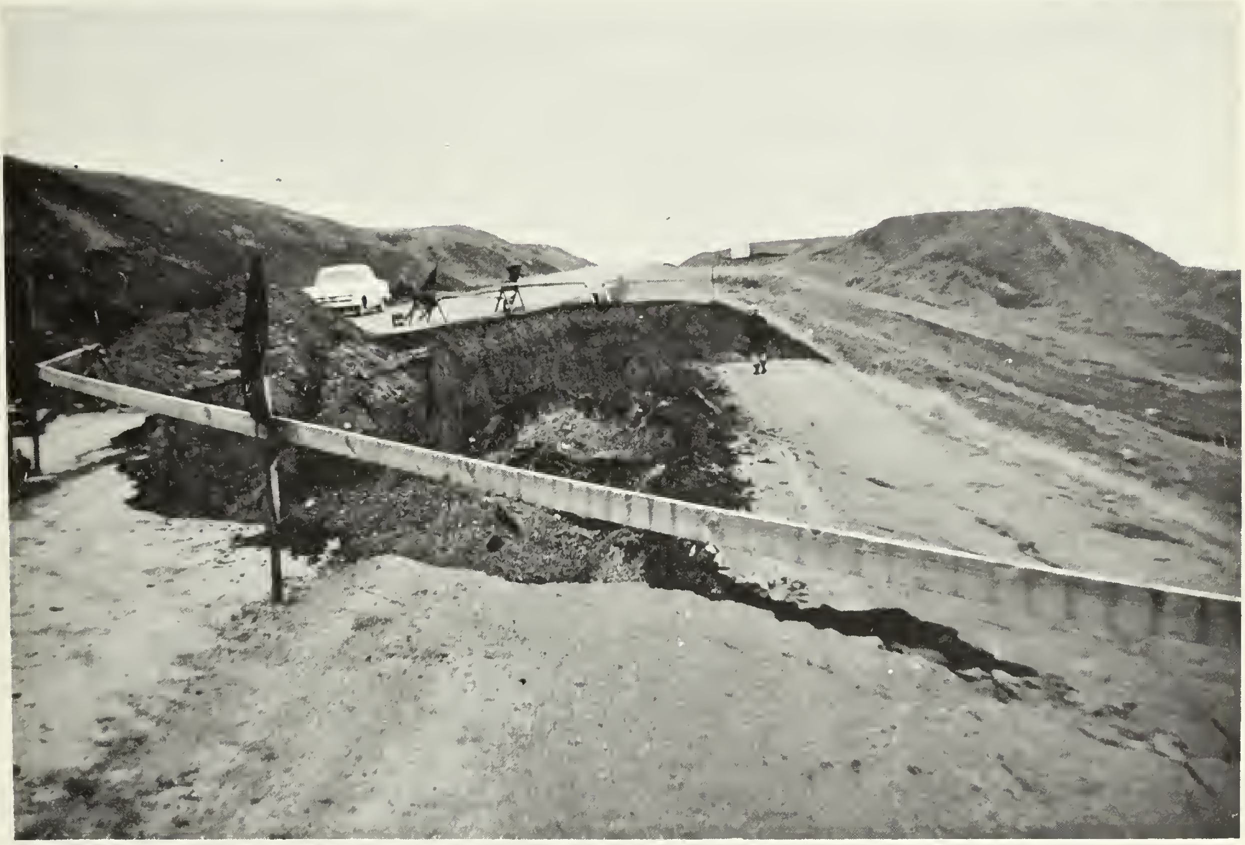


MONTANA
COUNTIES & COUNTY SEATS

SLIDE NUMBER A-8

Plate No. 1

Havre Slide No. 1



Note condition of roadway after the failure. This steep shear plane at this slide head is very typical of rotational landslides.

Landslide No. A-8

Havre Slide No. 1

LANDSLIDE EVALUATION FORM

1. Location County Hill Township 32N Range 15E Sec. 5

Highway U. S. 2 Project No. Immediately west of Havre F 95(9) Station 483+34 to 484+85

2. Dates of study May, 1955

Remarks concerning general weather and rainfall/snowfall conditions during the time of study.

In 1955 - 32" of snow melted and saturated the thawed hillside just prior to the landslide.

3. Date of origin Unknown Date Movement Ceased 1955

Most active season 1955

Relative activity (speed, etc) from origin to present See, Plate No. 1, page 55.

In 1955 the mass had a measured movement of + 1.0' per day despite the placement of a berm during this failure.

4. Physical dimensions Approx. 150 ft. x 100' ±

Approximate yardage involved 10,000 to 15,000 cubic yards

Height of cut/fill slope 30' ± fill on left side, 10'± cut on right

Angle of cut/fill slope Fill slope of 1½:1, Cut slope @ 1½:1 or steeper

5. Water table, rainfall information, and remarks concerning the effect of water in the unstable condition In addition to the surface saturation mentioned above, ground water was available following the top of the shale which is overlain by clayey glacial till. The ground water was measured 3 days after the sliding at 1.5' to 5.0' depths.

6. Description of rock or soil types involved in the slide and in the adjacent area

Bed rock in this area is Bearpaw shale, which is overlain by the weathered shale plus glacial till. The engineering classifications of the foundation soils were A-6(10) to A-7-6(18).

Havre Slide No. 1

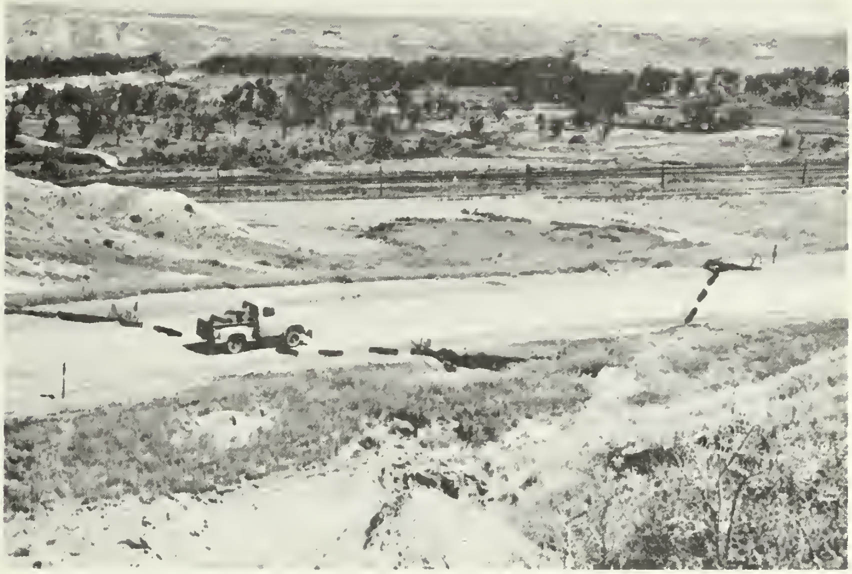
7. Probable Cause: Clay soils such as the foundation soils here involved are very sensitive to moisture changes; placement of a cut-fill section over these materials over-loaded the saturated foundation material and caused this failure. Sub-surface drainage on the uphill side was not installed and the fill was not properly keyed into the pre-existing slope. The normal ground water drainage was disrupted by the fill and this helped saturate the foundation and fill.

8. Curative Measures: A deep drainage ditch was installed on the uphill side and perforated drains installed to intercept surface and ground water entering the foundation and fill materials. Portions of the fill were replaced with some granular materials and keyed into the hillside. This area is now considered stable.

9. Data for avoiding similar situations in the future: During preliminary soil surveys, where clays are encountered on hillsides, special consideration should be given to the stability of the foundation. Examination will disclose the possibilities of ground water problems. Provision must be made to provide sufficient surface and sub-surface drainage to avoid saturation of the fill or foundation.

Plate No. 1

Havre No. 2 Slide



Note outline of the intersection of the shear plane with the surface. Temporary gravel surface covers this in the travelway, but dashed lines outline the approximate location.

Uphill ditch; note tension cracks near left center of this picture. Failure is toward the right. Darker portion in ditch bottom is free water.

Landslide No. A-8

Havre No. 2 Slide

LANDSLIDE EVALUATION FORM

1. Location County Hill Township 32N Range 15E Sec. 5

Highway U. S. 2 ±500 feet west of 1955 failure. Station Immediately west of Havre City Limits

2. Dates of study July, 1965

Remarks concerning general weather and rainfall/snowfall conditions during the time of study.

Ground water contributed to this failure and was observed flowing on the contact of shale with overlying glacial till.

3. Date of origin July, 1965 Date Movement Ceased July, 1965

Most active season Same

Relative activity (speed, etc) from origin to present No records of measured sliding velocity are available, however, the failure would be classified as fairly rapid. (See Plate No. 1, page 58)

4. Physical dimensions 150 feet and 200 feet in lateral extent

Approximate yardage involved ±30,000 cubic yards

Height of cut/fill slope Fill approx. 20' - 30', cut under 10'

Angle of cut/fill slope Fill @ 1½:1, Cut somewhat steeper

5. Water table, rainfall information, and remarks concerning the effect of water in the unstable condition Ground water at or near the shale-till contact followed old drainage channels cut into the shale prior to the glacial till placement and caused fill and foundation saturation.

6. Description of rock or soil types involved in the slide and in the adjacent area.

Bearpaw shale of Upper Cretaceous age is overlain by glacial till which is mainly silt with sand lenses and clay seams. The upper surface of the shale was weathered and eroded by water into channels prior to till sedimentation.

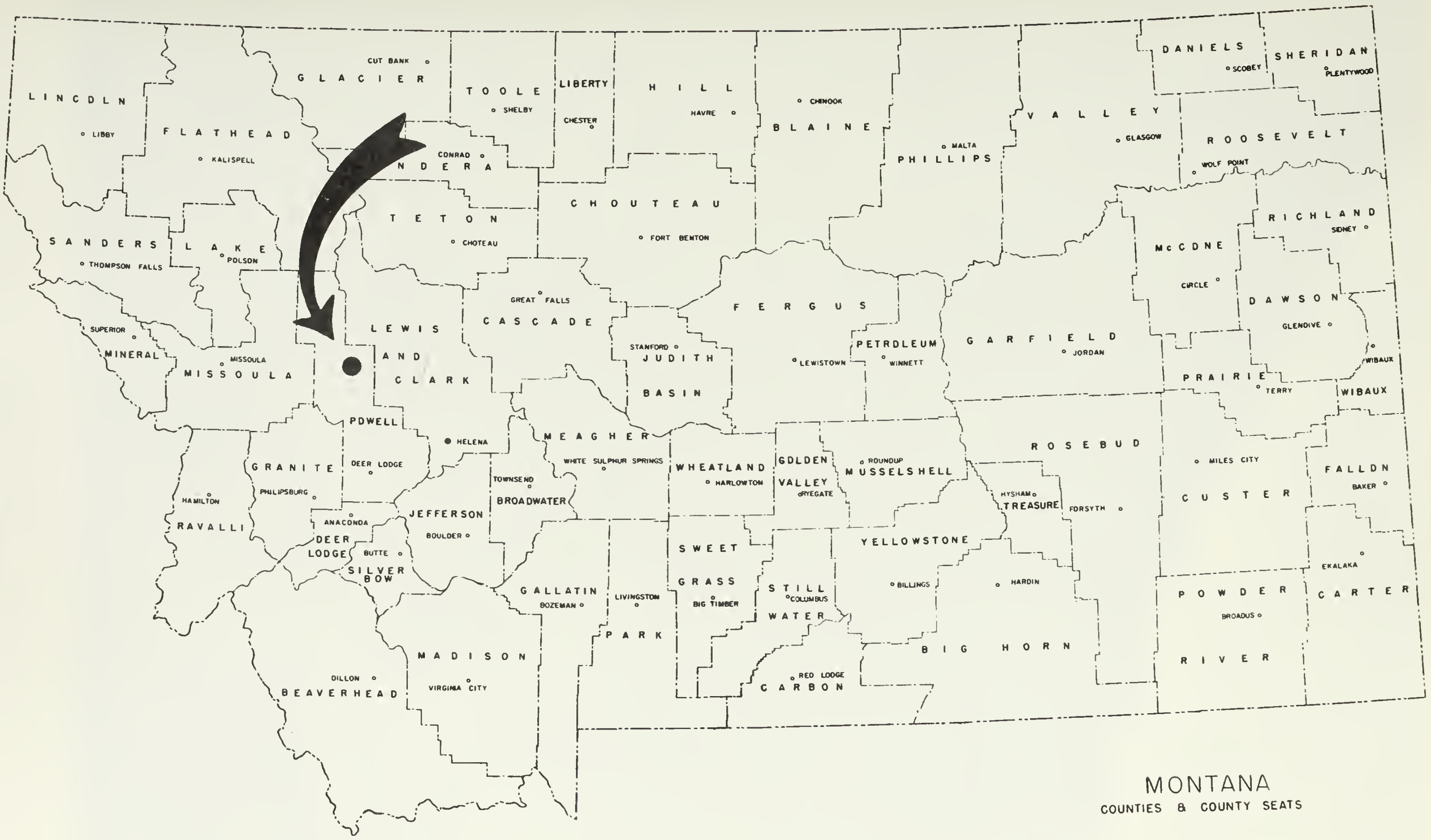
Havre No. 2 Slide

7. Investigation and Cause: The area was examined and geologically mapped in July, 1965. The conclusion drawn from this investigation indicated that ground water draining from the south, toward the failure zone, caused saturation of the silt and clay foundation with subsequent failure. This ground water recharged from normal precipitation and also from leakage of two artificial ponds at the Havre Fairgrounds just south of the failure zone.

8. Curative Measures: The recommendation was made to install a perforated drain below the slide plane along the uphill side. This would intercept the ground water and help keep the clay foundation dry. In addition a counter-balance was recommended together with sub-excavation of the clayey fill materials and replacement of these with granular embankment.

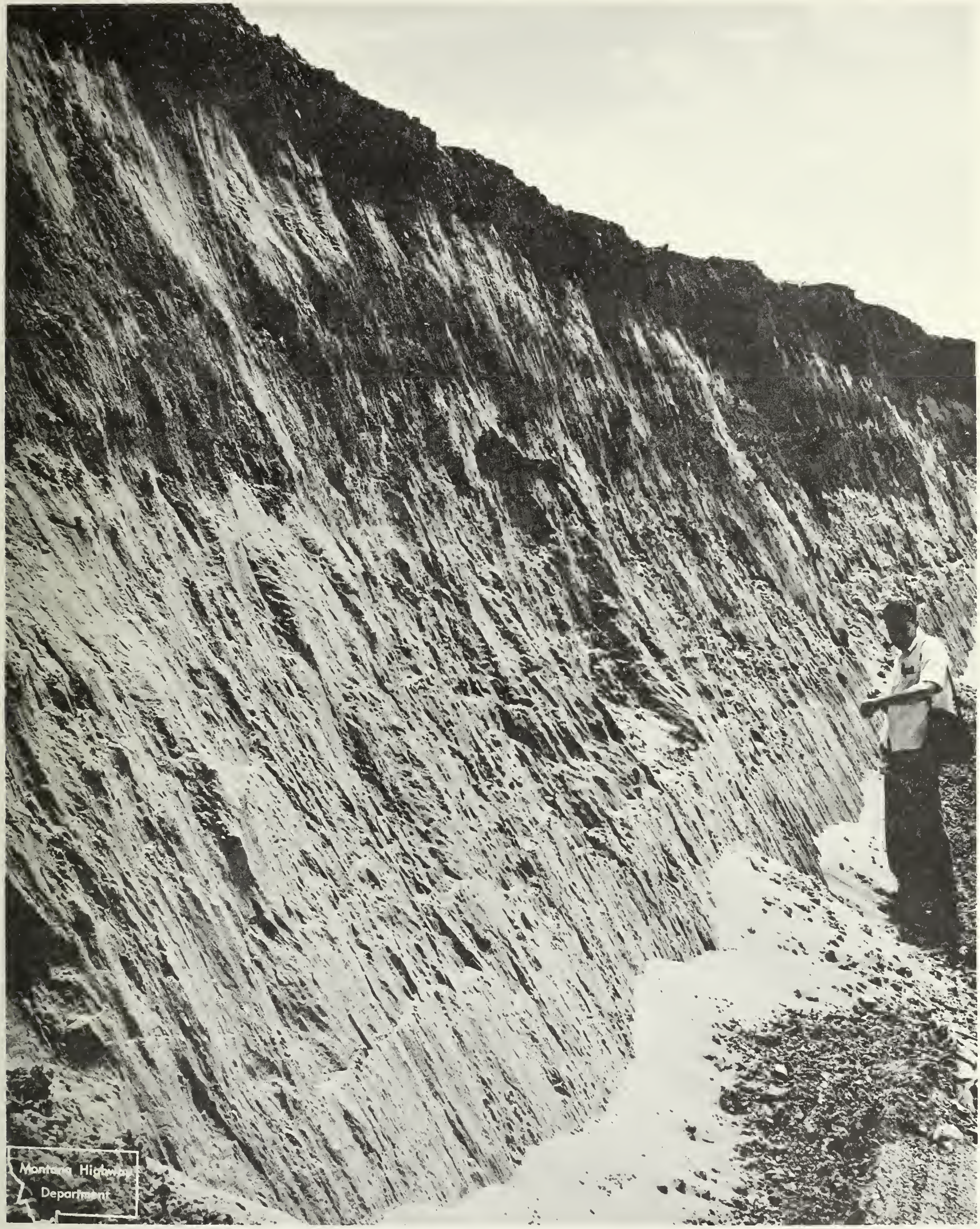
9. Data for evidence of similar conditions in the future: See recommendations for recognition and avoidance of similar situations in the 1955 slide report near the same location. (Havre Slide # 1)

OVANDO LANDSLIDE



MONTANA
COUNTIES & COUNTY SEATS

SLIDE NUMBER A-9



Ovando Slide
View of the head scarp; note the well developed slickensides.

Landslide No. A-9

Ovando Slide

LANDSLIDE EVALUATION FORM

1. Location County Powell Township 14N Range 11W Sec. 23
±10 Miles east of Ovando
Highway Mont. 20 Project No. F 267(8) 8-26 Station 2440± to 2449±

2. Dates of study June 27 & 28, 1962 - Preliminary investigation
June 3-14, 1963 - Resistivity study & auger holes.

Remarks concerning general weather and rainfall/snowfall conditions during the time of failure.

Heavy rainfall during late May & early June. River moderately high.

3. Date of origin June 24, 1962 Date Movement Ceased June 30, 1962

Most active season Total movement in above time (to date)

Relative activity (speed, etc) from origin to present Inactive since June 30, 1962. The failure occurred fairly rapidly with the initial movement in excess of 6 feet per day. (See plates 1 and 2, page 64 and 66)

4. Physical dimensions 250' X 700' ±, shallow circular arc.

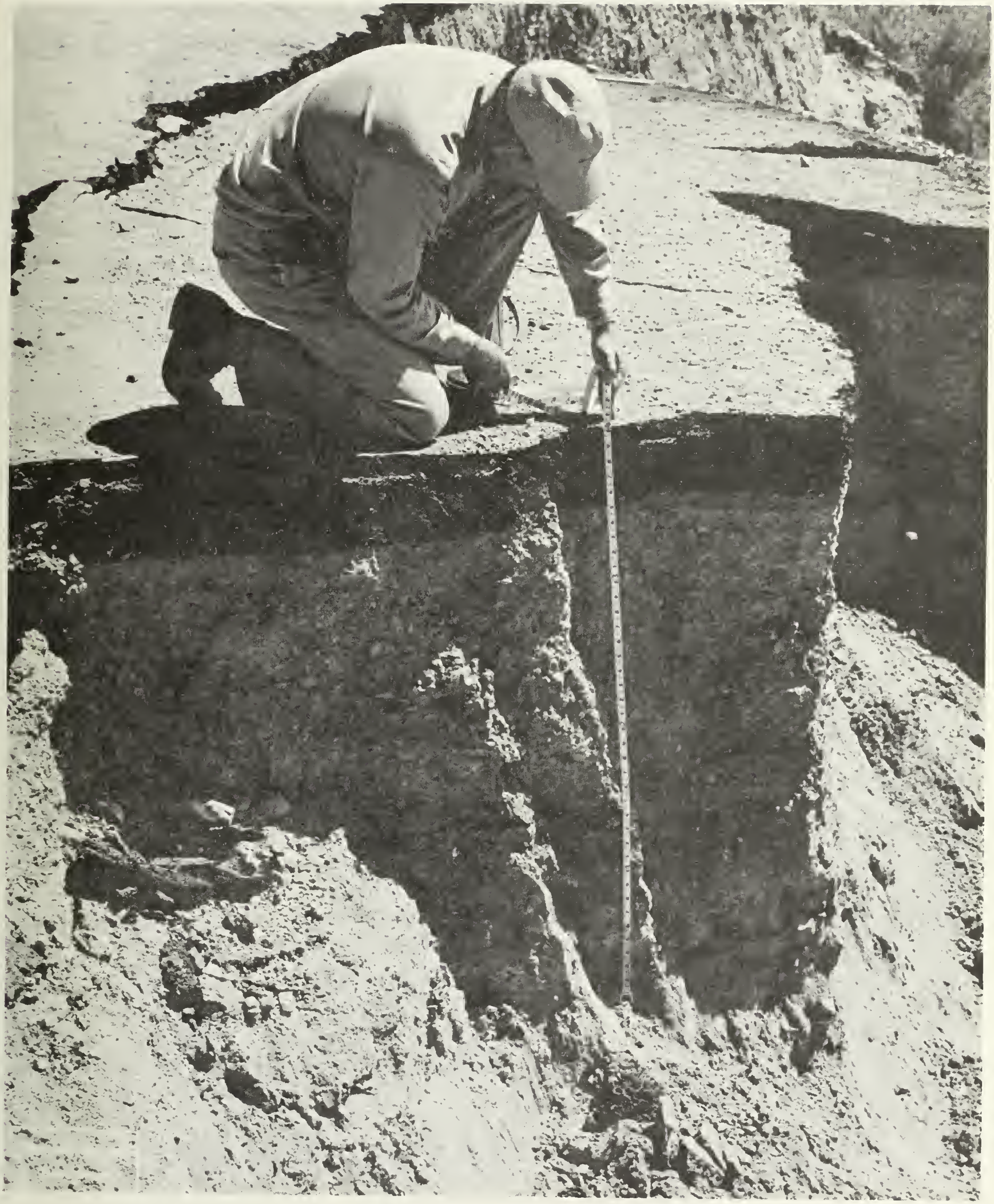
Approximate yardage involved 60,000 ± cubic yards

Height of cut/fill slope 10 ft. ± from 1955-1962; 2½ ft. added in April & May, 1962.

5. Water table, rainfall information, and remarks concerning the effect of water in the unstable condition Water table at a depth of 40 feet, which is well below the failure surface but was probably higher at the time of failure.

6. Description of rock or soil types involved in the slide and in the adjacent area

River deposited silty sand and sand over silty clay and clay. At considerable depth, well below the failure, is argillite bedrock.



Ovando Slide
Note granular fill materials above old temporary surfacing, with fine gravel and sand below.
Plate No. 1

Ovando Slide

7. Investigation: The original investigation included geologic study, estimation of the failure surface and resulted in a recommendation to move toward the north, away from the landslide. This move was accomplished.

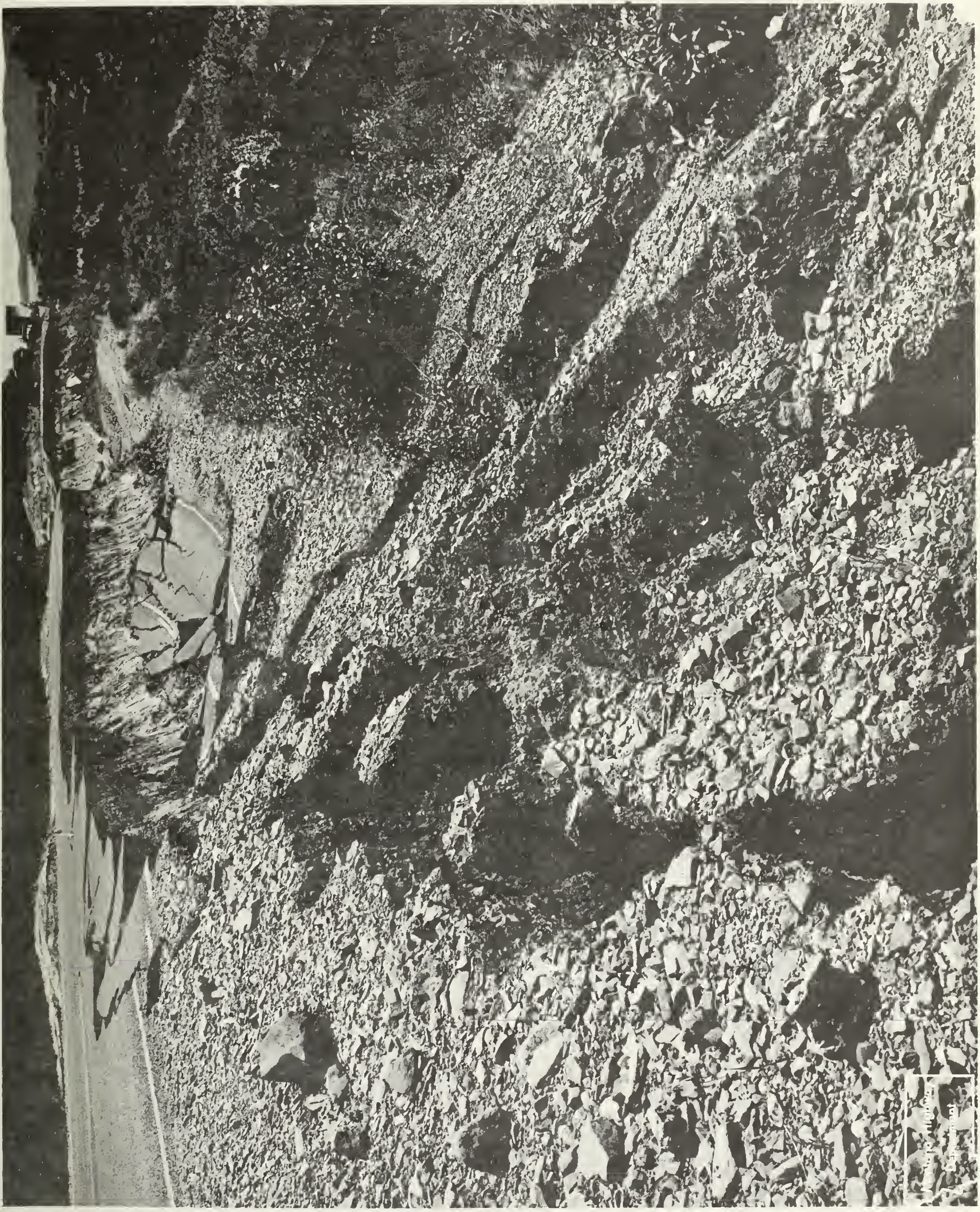
   The second investigation included a resistivity survey to locate the slide plane plus hand auger holes to recover samples and substantiate the resistivity data.

8. Probable Causes: The fill was placed in 1955 and temporary surfacing installed. Just prior to the landslide development, 2½ feet of material was added to this fill section and permanent surfacing placed. (See the accompanying photos, plates 1 & 2, page 64 and 66.

   Adding this weight evidently activated a primed landslide situation. The materials were saturated by rainfall and also by the Blackfoot River which adjoined the toe of the slide.

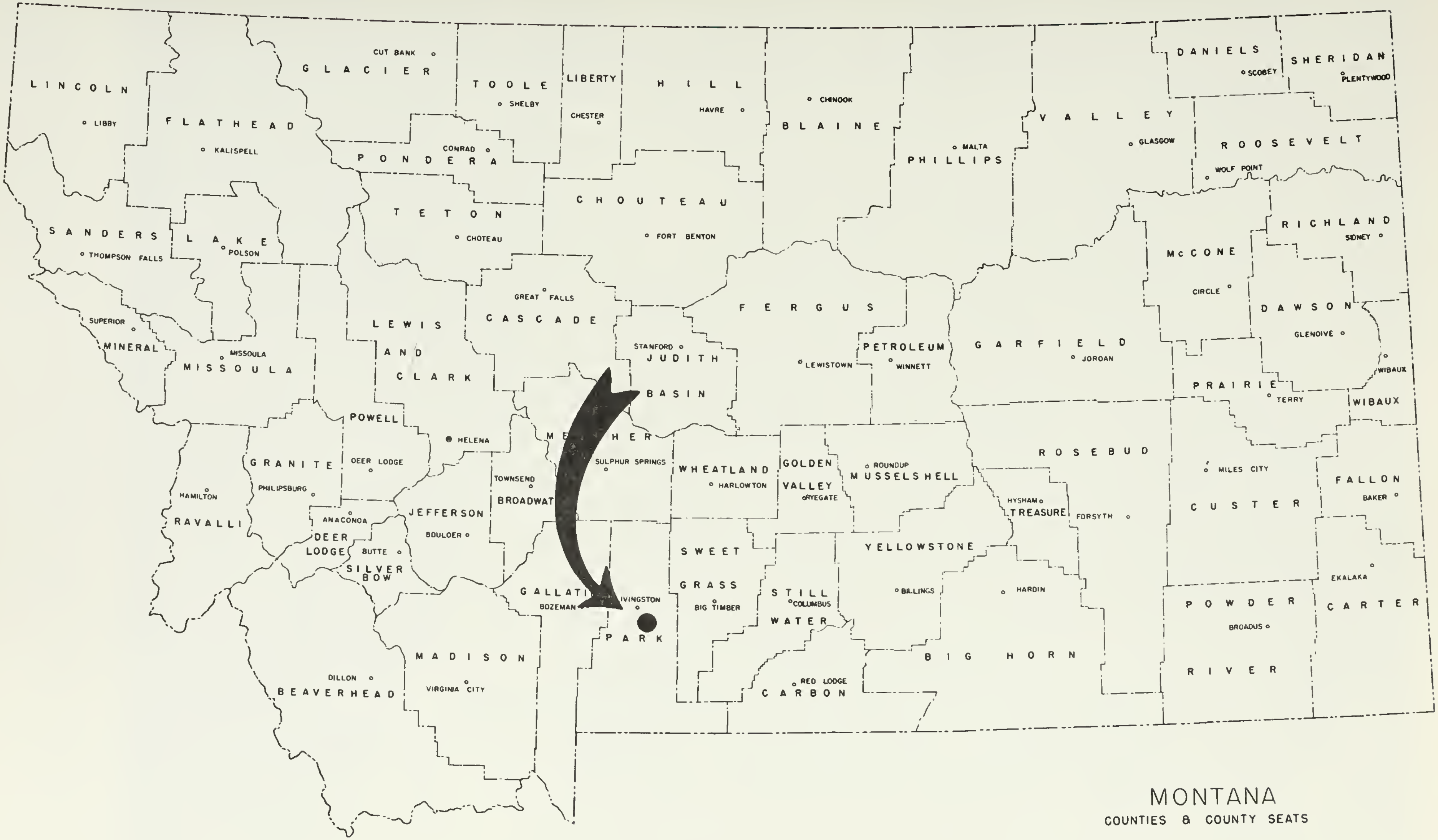
9. Data for avoiding similar conditions in the future: Normal soil surveys would not detect this situation as the surficial materials were granular and the water table deep. Possibly a detailed, deep reaching foundation survey would indicate the possibility of this slide developing.

   Only a complete preliminary investigation which would include drilling, soil testing (including shear analysis) and complete geologic mapping would indicate this dangerous situation.



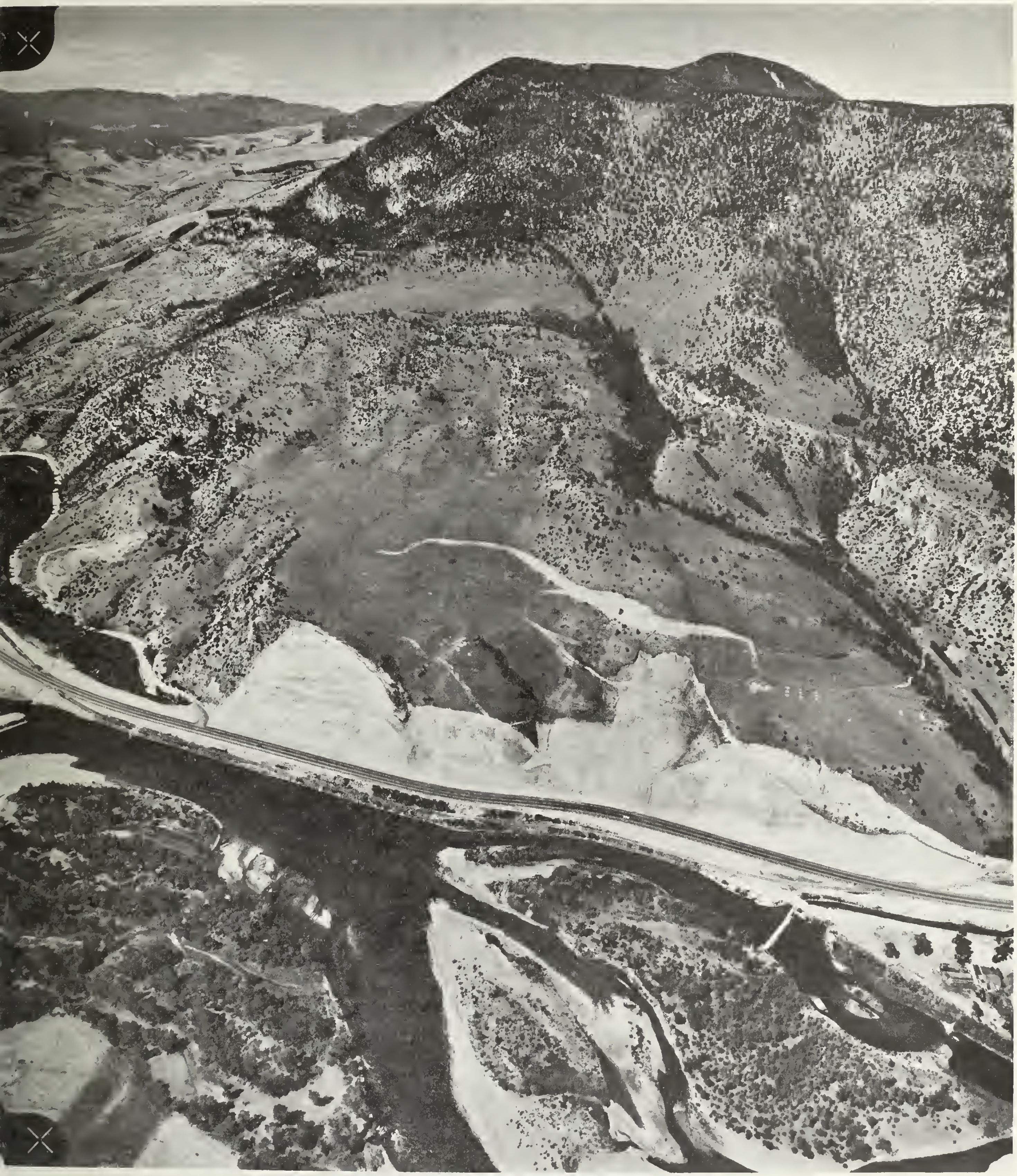
Ovando Slide
Note - general highway condition, shear surfaces and tension crack development in foreground
Plate No. 2

LIVINGSTON LANDSLIDE



MONTANA
COUNTIES & COUNTY SEATS

SLIDE NUMBER A-10



Oblique view looking south-west of entire landslide area. Note approximate outline of Pre-existing landslide marked with dashed line. Details of existing failure zone are shown on blowup, Plate No. A

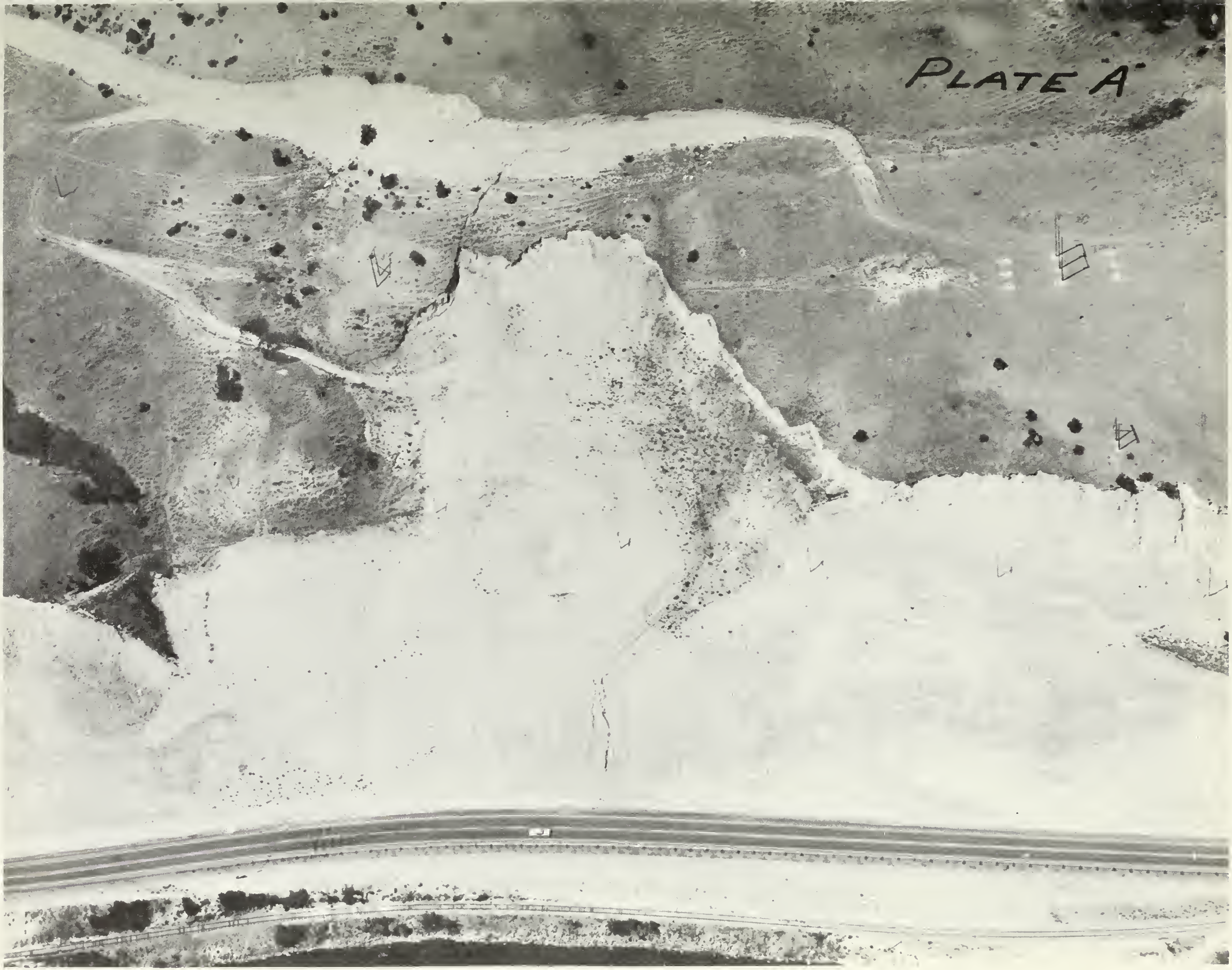
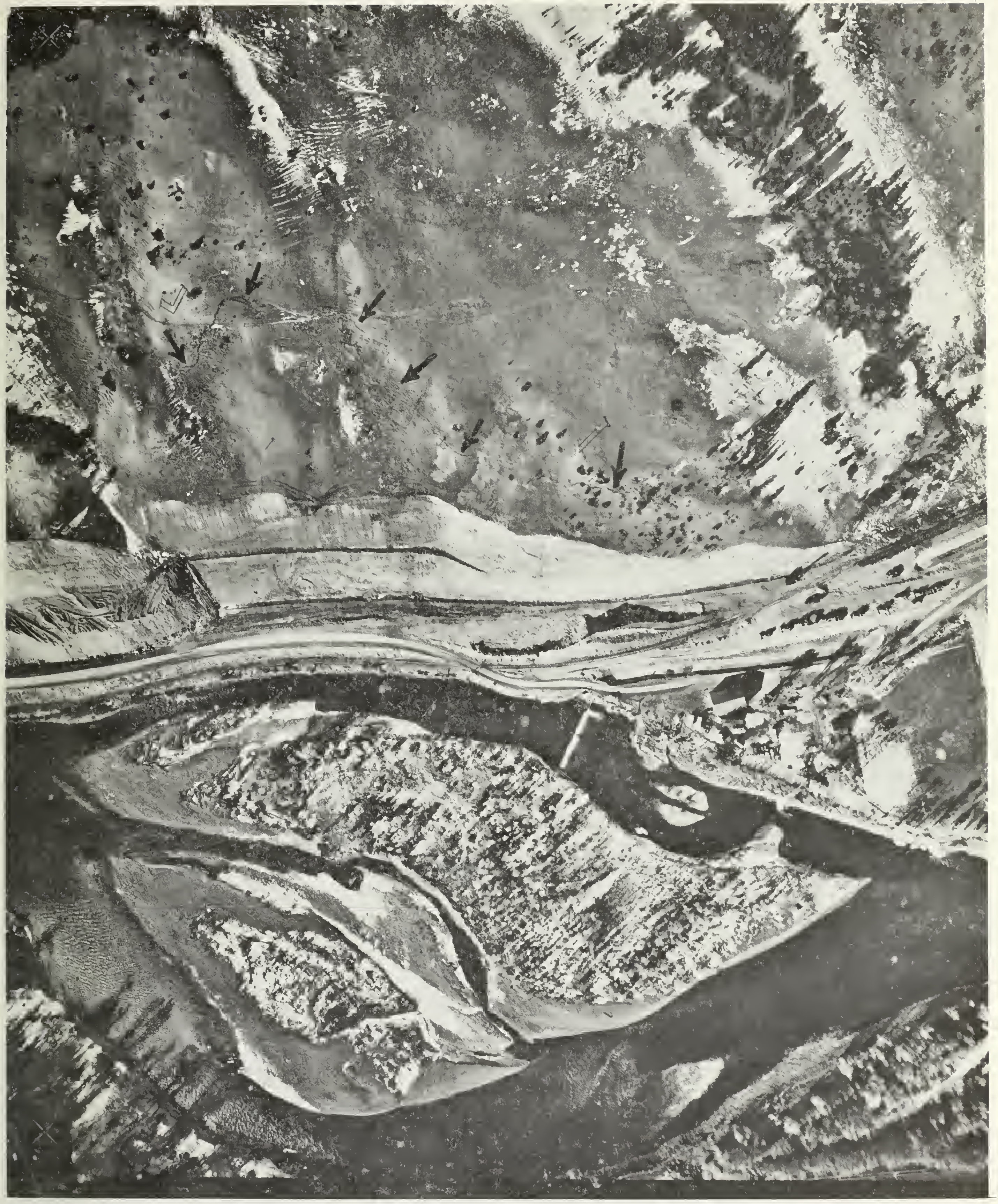
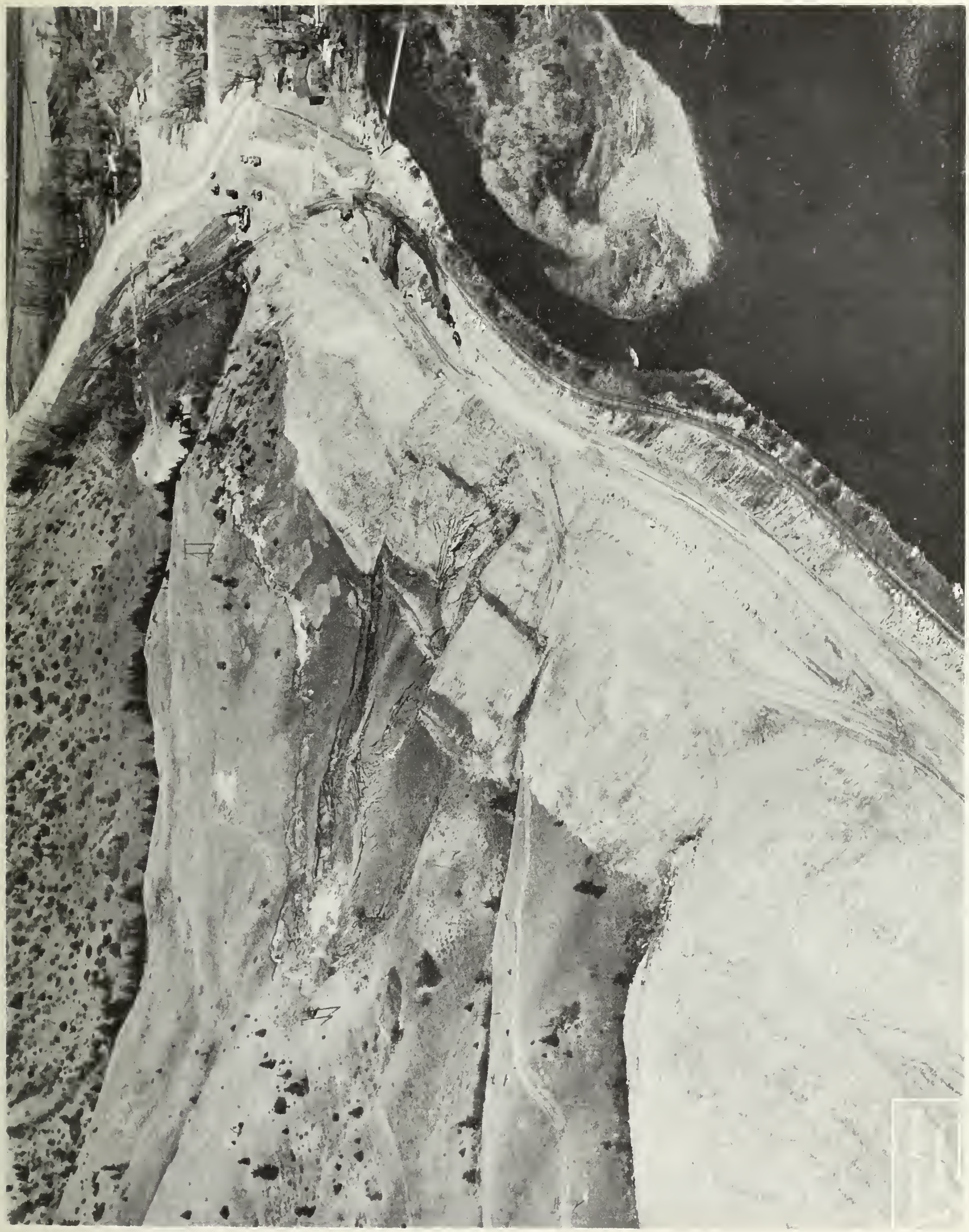


PLATE A

Livingston Landslide

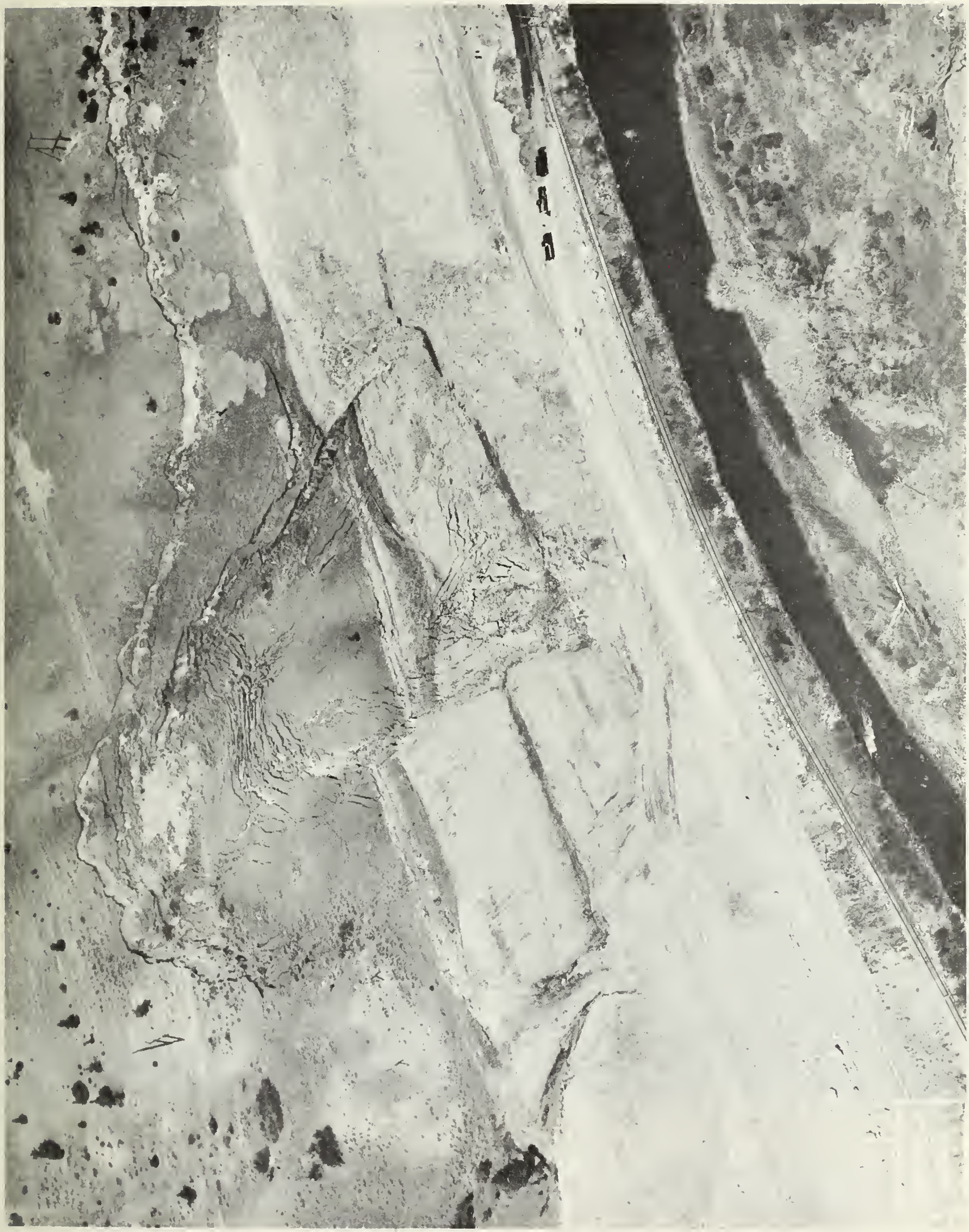


Vertical of general area taken before the large failure. Note arrows pointing to Tension Cracks. Picture taken Jan. 20, 1965.

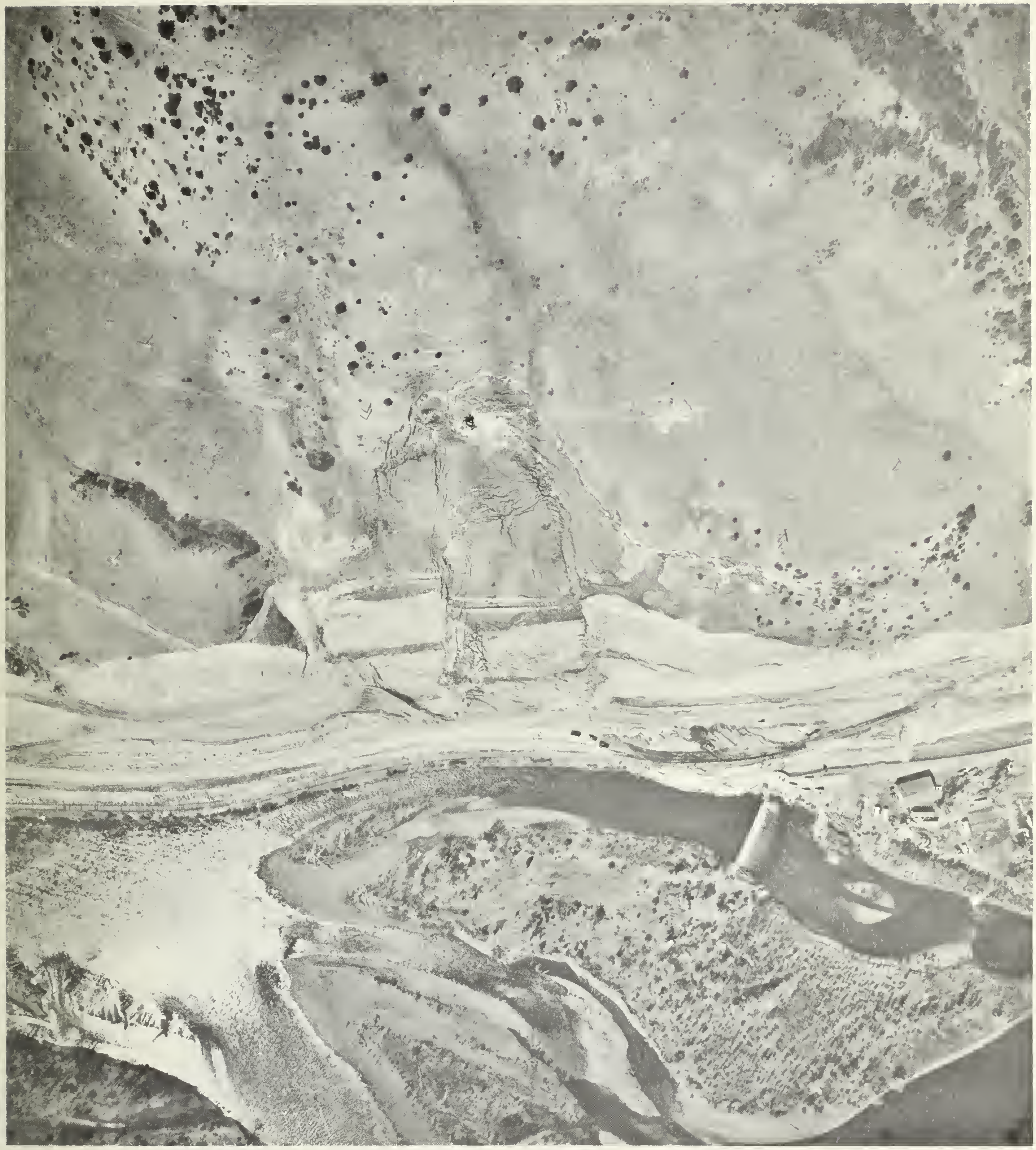


Oblique view toward north of failure, picture taken 4-27-65.

Livingston Landslide

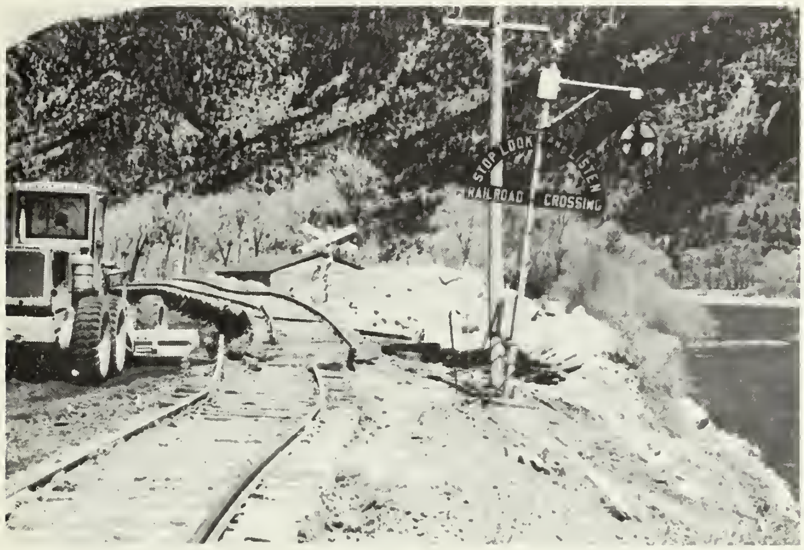


Oblique view toward west of failure, picture taken 4-27-65.

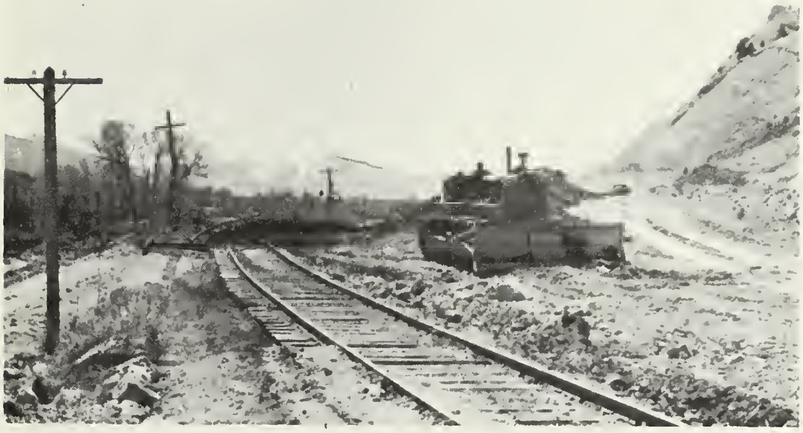


Vertical view of failure taken April 27, 1965. Note development toward the right in previously stable cut slope. Note excavation in vicinity of the diversion structure which relieved the pressure on it.

Livingston Landslide

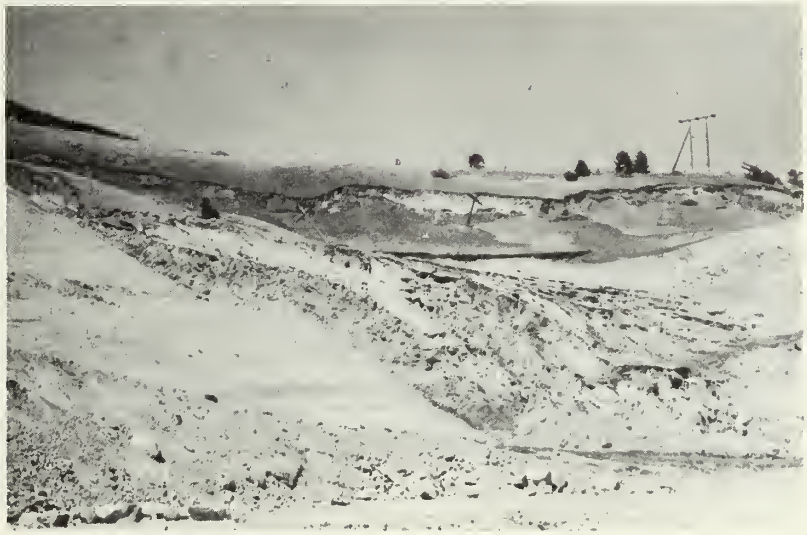


Railroad dislocation near south edge of failure zone. Picture taken April 22, 1965 looking north.

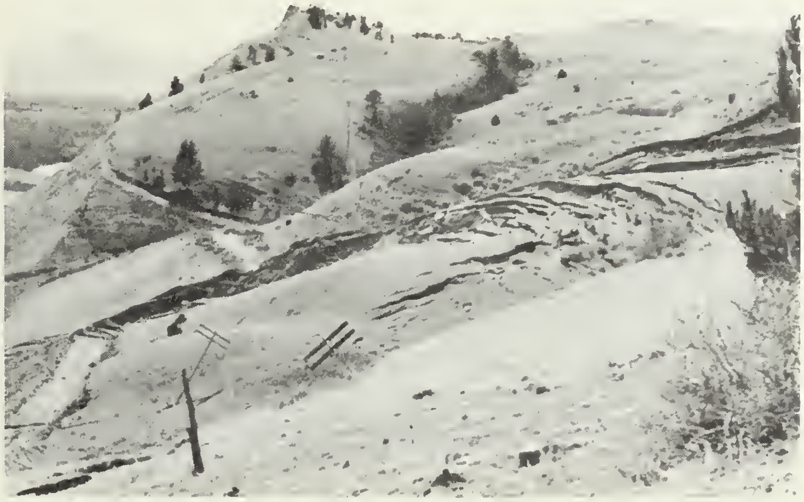


North end of railroad dislocation, picture looking south (4-22-65)

Livingston Landslide

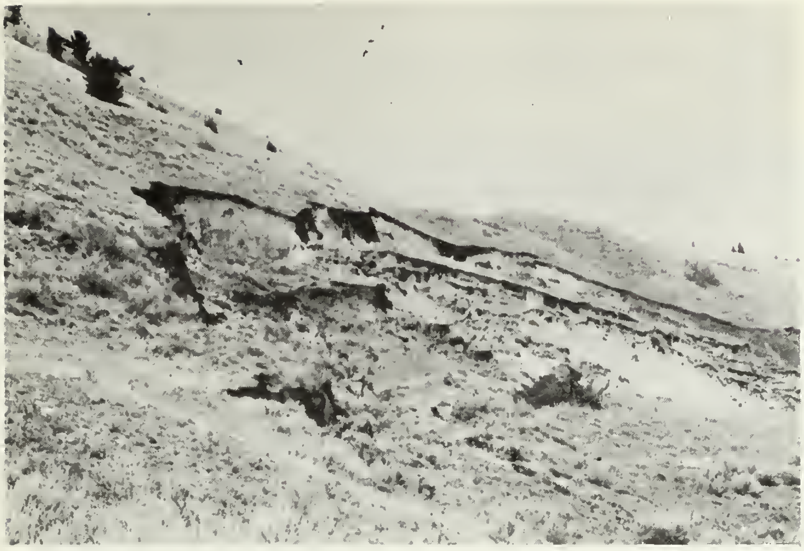


View across the middle of the landslide looking north from the railroad grade. (4-22-65)



View across the middle of the landslide looking south from approx. elevation of the head.

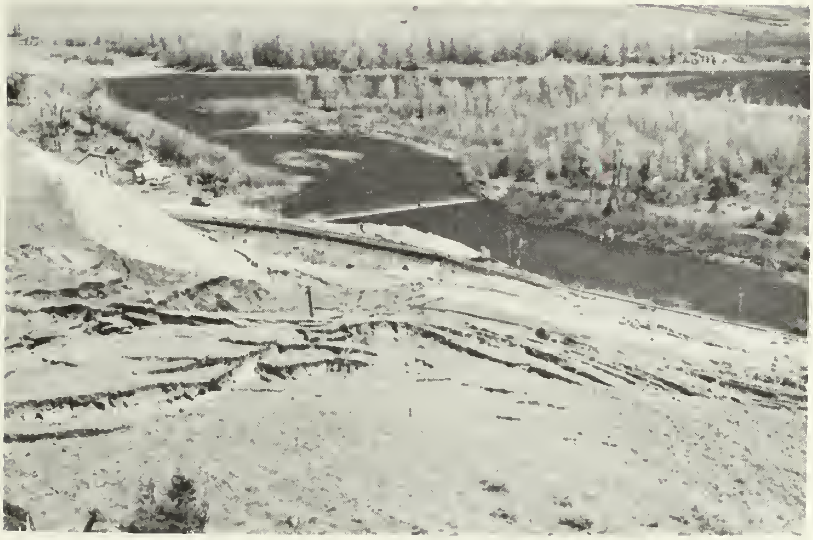
Livingston Landslide



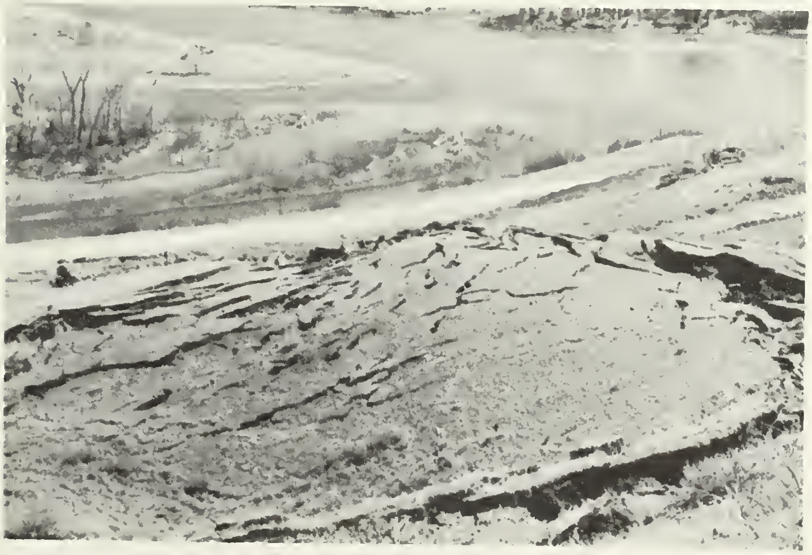
View looking north at the head of the slide. Note steepness of hillside above this initial failure. The head is creeping further to the left now.

View looking south of same area. (4-27-1965)

Livingston Landslide



View from top of sliding mass toward the north east. Note diversion dam in the Yellowstone River which was threatened by one landslide. (4-27-65)



General portion of the sliding mass, taken from the top scarp. Note bulge.

LANDSLIDE NO. A-10

Livingston Landslide

HISTORY

The subject landslide occurred during construction of Project F 217(11), Gardiner-Livingston at Station 1570 ±, approximately 5 miles south of Livingston, Montana. The first major movement occurred during the night of April 21, 1965 but tension cracks developed at least 4 months previously. The project was under construction and the contractor was engaged in excavation of the large cut sections in this vicinity when the failure occurred. Subsequent remedial actions included the following:

A Trenching to relieve pressure on a diversion structure in the Yellowstone River.
B. Relocation and regrading of the railroad bed.
C. Redesign of the cut slopes to remove driving weight.
D. Redesign of the railroad and highway grades and alignments to avoid cutting more than necessary into the toe area.
E. Establishment of a surface drainage ditch.
F. Drilling of horizontal holes into the landslide to attempt to drain water from critical areas.

Investigation of the landslide during 1965 included the following:

A. Eight vertical drill holes in the slide area.
B. Four horizontal drill holes in the slide area.
C. Electrical resistivity profiles.
D. Geology mapping.
E Stability studies by Butte District personnel.

The remedial actions were sufficiently effective to allow completion of the project.

During the spring and summer of 1966, some movement was noted by Division field personnel who initiated a request for further study. The study was conducted, with Highway Research Funds, during the late summer and fall of 1966, and included the following investigative methods:

(a) Establishment of survey points to determine the rate and amount of movements.
(b) The drilling of four additional vertical test holes to extend the information obtained during 1965.
(c) Topographic mapping of the entire area of instability.
(d) Geologic mapping of the general area.
(e) Seismic study.
(f) Resistivity study.
(g) Stability analysis.

The data obtained are further discussed in this report.

Physiography and Climate

The landslide lies near the north edge of the Beartooth mountains, immediately west of the Yellowstone River.

The Yellowstone River flows north from Yellowstone National Park, turning east along the north slope of the mountains at Livingston, five miles north of the slide area. The landslide lies on an east facing mountain slope between elevations 4550 ft. and 6100 ft., with the original toe below the existing strand line of the Yellowstone River. The average rainfall at Livingston is about 15 inches (39 year record) with the month of May yielding the highest average moisture during the year.

The Yellowstone River discharge at Livingston has averaged about 3600 second-feet of water for 23 years record period. During the same period the maximum discharge of the Yellowstone River at Livingston was 30,600 second-feet on June 20, 1943 and the minimum discharge was 590 second-feet on January 22, 1940.

Geology

The landslide lies just north of the Suce Creek thrust fault which dips toward the north under the landslide. In the immediate vicinity of the slide, the Cambrian age Grove Creek fm is thrust over the Cretaceous age Mowry fm. The upper plate of the thrust, bordering the south edge of the slide, has been folded and overturned toward the south. Thus the south edge of the landslide borders overturned Cambrian age rocks. Along the northwest and north edge of the landslide normal sections of Paleozoic rocks border the slide. These are generally dipping 15^{o} to 26^{o} toward the north or north-northwest.

The original landslides covered approximately one square mile and contained very large blocks of limestone together with rubble composed of crushed limestone, shale and large quantities of silt and sand sized, relatively non-plastic materials. Topographic study of the landslide seems to indicate that at least two large, semi-rotational movements have taken place in the past, each of which is characterized by large limestone blocks on the east edge, and relatively flat, hummocky ground in back of the limestone blocks toward the west. The head of the top slice lies at an elevation of about 6100 ft. with the toe at approximately 5300 ft.; the second slice lies between 5300±ft. and 4550 ft. The toe of the lower slice probably lies well below this mapped elevation. These slices become more confused toward the southeast. There are numerous small interior drainage areas on the surface and very poorly developed surface drainage patterns throughout the landslide. Several springs were found.

Near the ridge tops at approximate elevations 5200 ft. and 5400 ft., many trees are bent outward and bowed toward the east. This has probably been caused by soil creep but the lack of developed surface drainage indicates that some if not all of the movement has taken place in recent time. Some evidence of a fault parallel to the valley bordering the landslide on the northwest was found but the fault could not be traced beyond the flat area at the head of this valley. Evidence seems to indicate that if this fault existed, the valley was developed prior to landslide

activity inasmuch as it is the only well developed drainage in the landslide area.

The existing problem area is in the lowest original slide which probably extended well beneath the present river elevation. The large limestone blocks bordering the existing failure zone attests to this as they are part of the original landslide debris. During drilling in 1965 and 1966, a remarkable lack of ground water was encountered. Some very small areas containing free water were found within the sliding mass but the present water table is near the elevation of the Yellowstone River.

Materials encountered in the existing slide are limestone rubble and silt, overlying shale. Several zones were encountered which are or have been sliding surfaces or faults. A vertical drill hole was drilled 400 feet southwest of the existing failure in a broad, shallow basin. No apparent movement has been noted in this vicinity within the time interval covered by this report, however, surface features suggested that this basin-type area might be a natural drainage furnishing water to the existing failure. This hole was drilled to a depth of 365.4 feet which is approximately 120 feet below river elevation and is the deepest hole that has been drilled at the site. Silt, limestone and shale rubble were encountered to a depth of 48.1 feet. Between 48.1 and 257.8 feet the hole penetrated extensively fractured limestone with interbedded layers of siltstone and shale. The limestone is brecciated locally and probable slip zones or former slide planes were noted between depths of 89.5 and 96.0 feet, and 252.0 and 257.8 feet, respectively. Within these zones the hard, competent limestone occurs as angular fragments within a matrix of shale and/or clayey silt, locally this mixture is cemented forming a breccia. Below a depth of 257.8 feet interbedded, dark gray-green shale and gray limestone were encountered to a total depth of the hole. Vertical bedding was noted at three different elevations in this material with a near horizontal attitude between these vertical zones. Occasional slickensides were also noted. The attitude of the shale and limestone indicated that this material had been subjected to intensive folding. Insufficient information is available to indicate the nature and magnitude of this geologic structure. Measurements made through perforated casing indicate the ground water level is at a depth of approximately

241 feet which is near the elevation of the Yellowstone River east of the drill site.

Landslide Discussion

The present landslide was triggered by construction of the highway as the pre-existing landslide toe was removed. While a major problem existed and does exist, it is apparent that the actual failure is a very small part of the pre-existing landslide zone. The portion which failed was saturated, at least in part, as witnessed by project, division and district personnel. Water and flowing silt was observed during and shortly after the failure at and near the toe area. Every indication pointed to a circular arc failure, as evidenced by the railroad displacement upward, and the shape of the failed surface. Some lower arcs developed, or the lower portion of the failure became planar because pressure was put on the diversion structure in the Yellowstone River well below the major failure. Trenching between the slide and concrete structure immediately relieved this pressure.

The landslide was described by the Resident Engineer, as follows:

> " The original movement, which produced a small 8" scarp occurred on or about December 3, 1964, when the cut was almost to grade except at the north end. The cut slope remained static through the winter. On April 7, 1965, the roadway between 1574 and 1579 heaved 2 feet. On April 19, 1965, a small crack was noted in the roadway at the north end of the cut with a heave of about 6 inches. The major movement occurred on April 22 and 23, 1965, and slowed on April 24th. The head of the slide moved vertically 6 ft. ± on the 22nd and 6 ft. ± on the 23rd. The total lateral movement in the roadway was 30 ft. ±. The failure from station 1574 ± to 1576+50 was toward and nearly perpendicular to project centerline, while from 1576+50 to 1580± the movement was northeast at about an angle of 45° as measured from the roadway alignment. The Northern Pacific tracks were displaced approximately 8 ft. east and 2 ft. upward."

This landslide moved very slowly during 1966 directly toward the highway as far as could be determined. Evidence of roadway heave was not found during 1966, although some movement had roughened the driving surface. The railroad grade has remained stable to date since the survey control points were established. The landslide head is migrating west and tension cracks have developed above the drainage ditch.

On May 12, 1967, the landslide became active and moved 50 feet + in approximately 12 hours. The failure was rotational and the toe emerged just over the roadway elevation in the cut slope. The material was removed from the roadway and temporary stability exists at the present time. The failure closely approximated Arc. No. 3 depicted in the stability analysis, see page 86.

Laboratory Testing

Materials in the existing landslide were classified by testing as A-1-a to A-4(8) with liquid limits of 17 to 34 and plastic indexes of NP to 9. Natural moisture contents ranged from 7.7% to 24.6% well below the plastic limit in most cases.

Shear Testing results varied greatly, but in general the unconfined shear strength value obtained from samples of 1033 lbs/ft.^2 could be considered representative of the ultimate unconfined shear strength. Triaxial testing indicated shear strengths of the same magnitude.

Resistivity Study - 1966

On November 2 and 3, 1966 two electrical resistivity surveys were made within the slide area. The resistivity instrument was constructed from a Bureau of Public Roads design.

These surveys were not adequate to evaluate the instrument or the applicability of this method of investigation.

Refraction Seismic Study

Seismic studies were undertaken prior to test hole drilling to determine where test holes should be drilled and to what extent this type of study could be used on this project. The results of this seismic study were about as anticipated.

This investigative method will not find the sliding surface unless this plane is directly over a material which possesses higher seismic velocity properties than the materials above the failure surface. This method is also restricted by depth limitations inherent in the equipment, and surface configuration of the area.

Stability Analysis

The stability analysis of this landslide was dependent upon the accuracy of determination of the following variables:

(a) Geometry of the failure and surrounding area.
(b) Water distribution.
(c) Physical characteristics of the materials involved.
(d) Nature and configuration of the failure surface.

The Livingston landslide is very complex because the existing failure zone is only a very small part of the pre-existing landslides. The materials involved in sliding are limestone rubble and silt. Shear strengths, in place density and natural moisture contents are dependent upon the relative amounts of silt and limestone fragments as well as the availability of water in the form of rain and/or snow and are, therefore, almost indeterminate. Laboratory shear strengths determined from samples of the silt were probably not representative of the mass. The existing shear strength failure has been progressively advancing the head toward the west. The shear strains are probably developing slowly at the present time. Surface runoff is no doubt adding weight by changing the moisture contents and thus the unit weights of the materials in the slide.

The preferred approach to a stability study in this situation is to assume a dynamic condition, with a safety factor of under 1.00. In this manner it is possible to determine _relative_ safety factors under certain forseeable conditions. This was the type study undertaken. The following assumptions were used in this analysis:

(1) The failure approximated a segment of a circular arc.
(2) The in-place density was 130 lbs/ft.^3.
(3) The existing safety factor on the failing area was under 1.00.

(4) The shear strength values were fairly constant and will remain so in the forseeable future.

With these assumptions several cross sections and trial arcs were inspected. (Refer to typical Section E-EE, page 86).

Arc No. 3 in Section E-EE was found to have a safety factor of 0.901 using shear strengths indicated by testing, i.e. Cohesion = 325 lbs/ft.^{2} with a friction angle of 15°. Arc No. 3 is believed to approximate the latest failure surface. The safety factor obtained bears out this assumption, as did the examination of the May, 1967 failure. It will be noted that Arc No. 4 had a lower safety factor. This is due to the geometrics of the area. The fact there has not been a failure along this arc indicates that either (1) higher shear strengths exist in the materials further west or (2) the so called "starting" friction has not yet been overcome. The safety factors were derived by the use of the following equation:

$$\text{Safety Factor} = \frac{\text{Resisting Forces}}{\text{Driving Forces}}$$

Where Resisting Force = C + [illegible] Tan Ø
C = Cohesion X Arc length
[illegible] = Stress normal to slip surface
Ø = Angle of Internal Friction
Driving Force = Stress tangential to the slip surface.

Conclusion and Recommendations

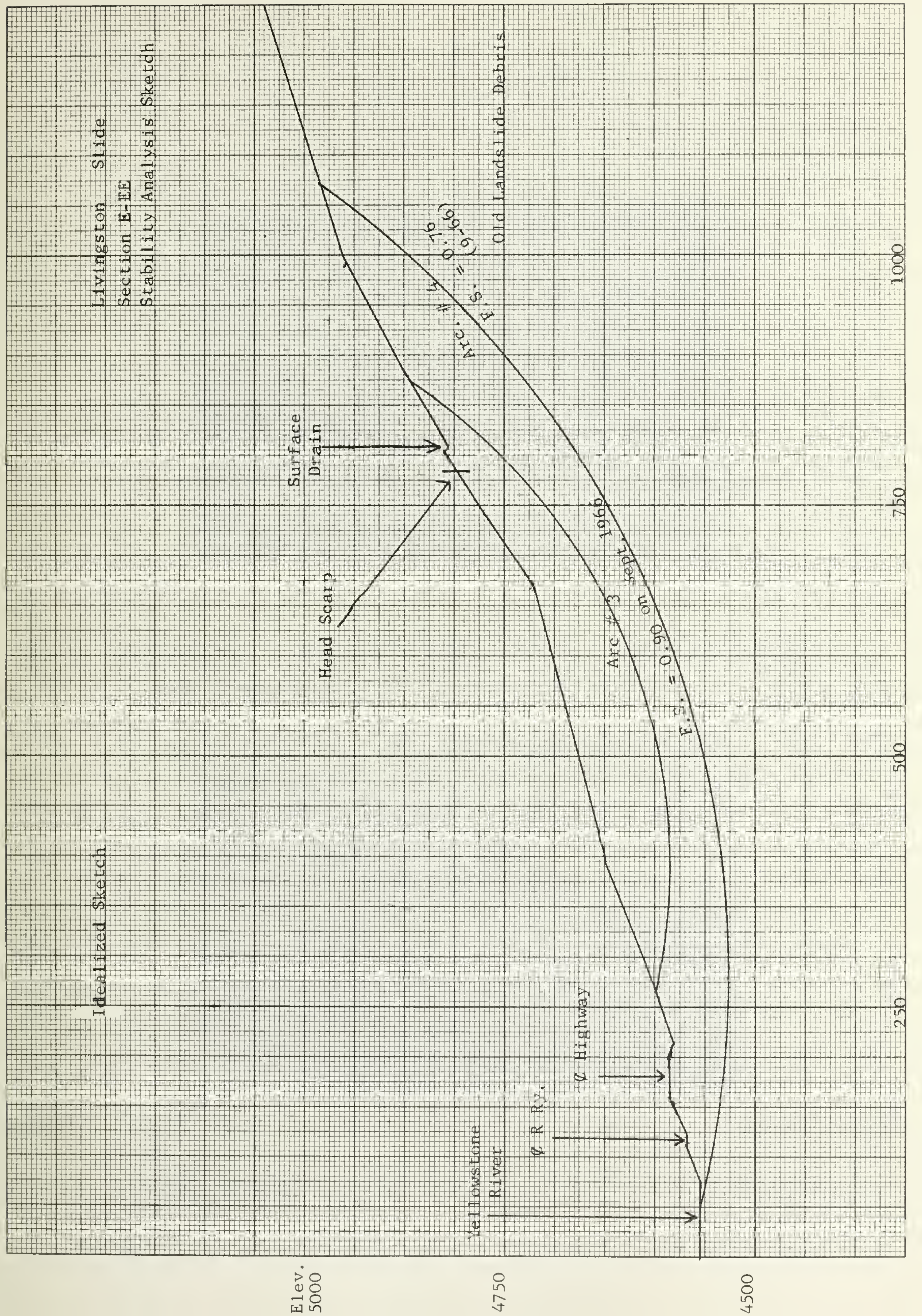
Because of the complexity of a small failure superimposed on a larger landslide, the lack of free water, the cost of further investigation and other factors; further investigation and positive curative measures were not feasible.

Possible corrective measures include the following:

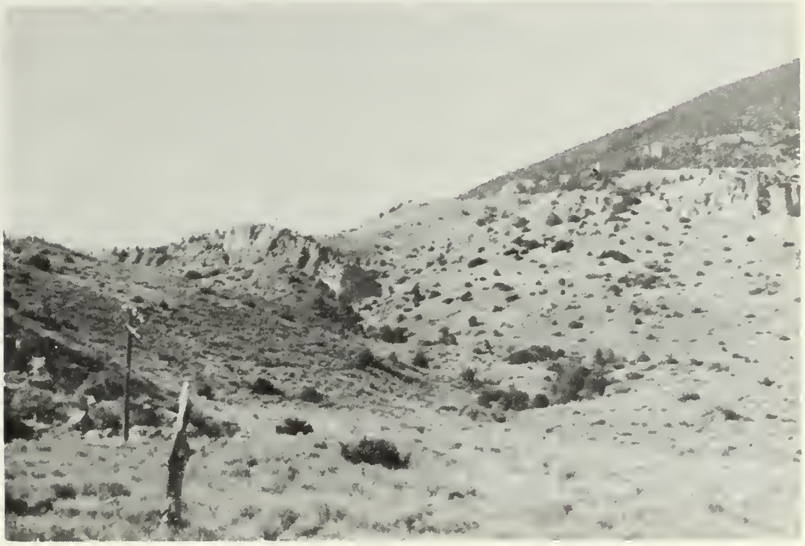
(1) Move away from the failing zone.

(2) Attempt to unload the failure head and/or load the toe (this was not considered possible because of the larger possible failure).

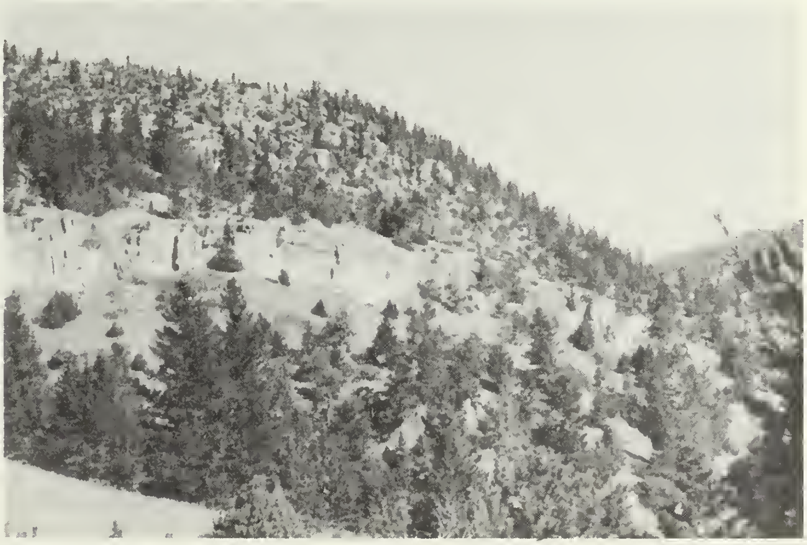
(3) Attempt to live with the failure in the existing location, as long as possible.


Livingston Slide
Section E-EE
Stability Analysis Sketch
Idealized Sketch
Old Landslide Debris
Arc. # 4
F.S. = 0.76
(9-66)
Surface Drain
Head Scarp
Arc # 3
F.S. = 0.90 on Sept. 1966
℄ Highway
℄ R Ry.
Yellowstone River
250
500
750
1000
Elev.
5000
4750
4500

Livingston Landslide



Topography on the Landslide, note limestone rubble, size of slide blocks, poorly developed drainage. Taken near center of landslide area looking south.



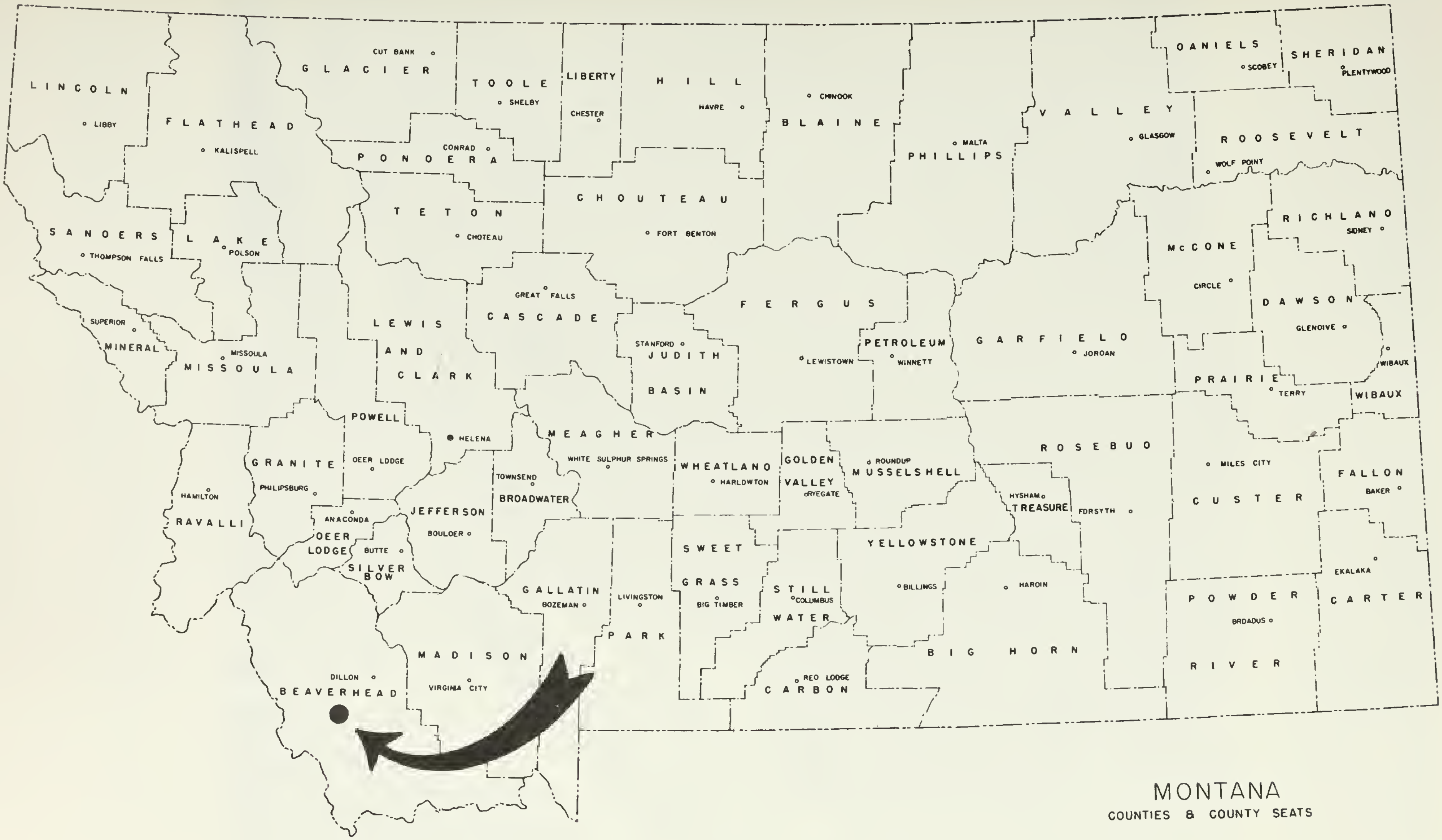
Outcrop of Bighorn Dolomite, taken on north west edge of the landslide area.

Livingston Landslide

Outcrop of Pilgrim Limestone at north edge of Landslide Area, Normal Section.

Outcrop of Pilgrim Limestone at south edge of Landslide Area, overturned section.

PIPE ORGAN LANDSLIDE



MONTANA
COUNTIES & COUNTY SEATS

SLIDE NUMBER A-11

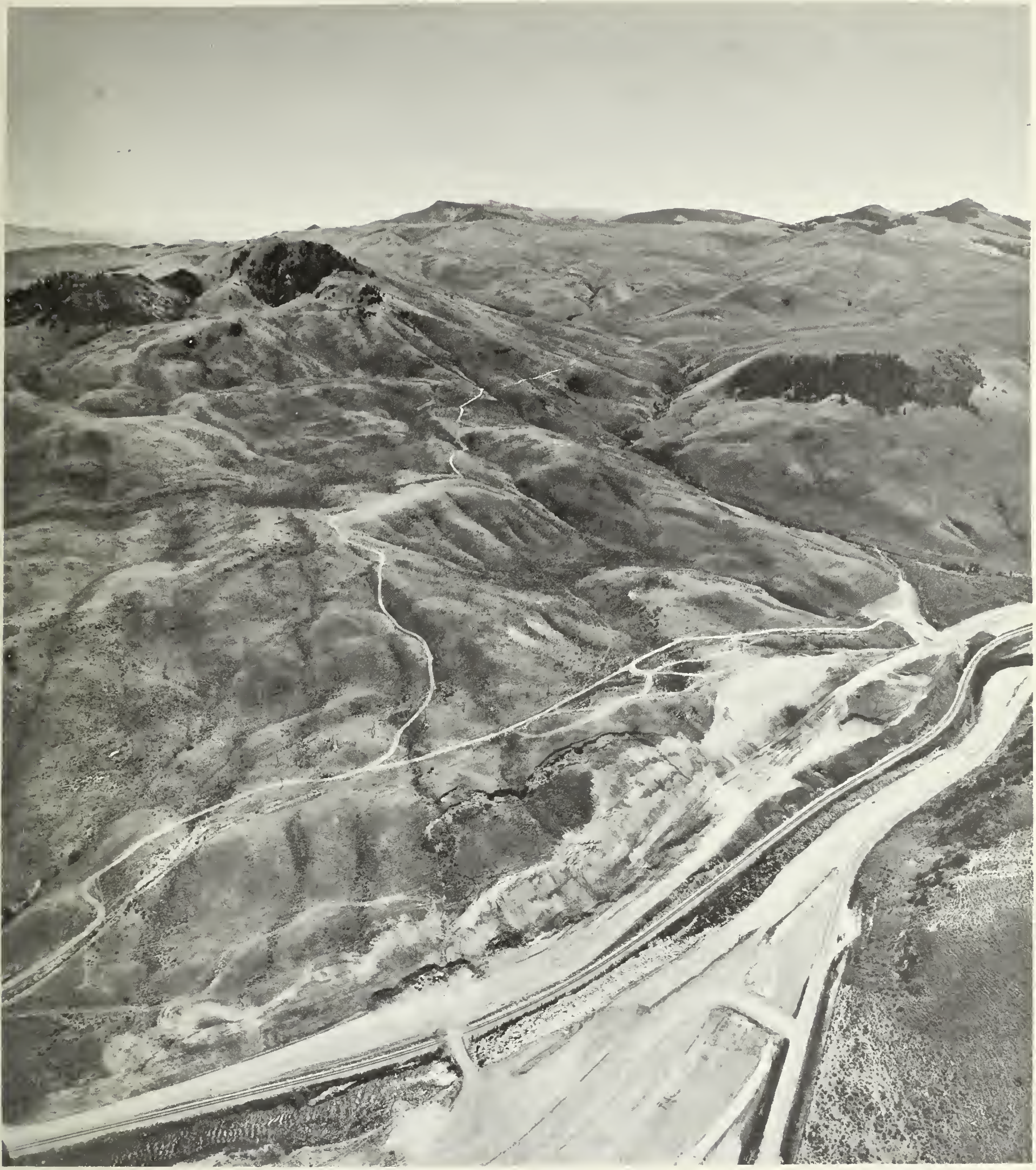


Figure 3-1, Oblique photograph taken July 16, 1965, looking to the southeast, showing the first landslide that developed in this area. The present landslide is located to the left of this movement in the area that was under construction at the date of this photograph. Note the small cut section that the railroad designed on their alignment.

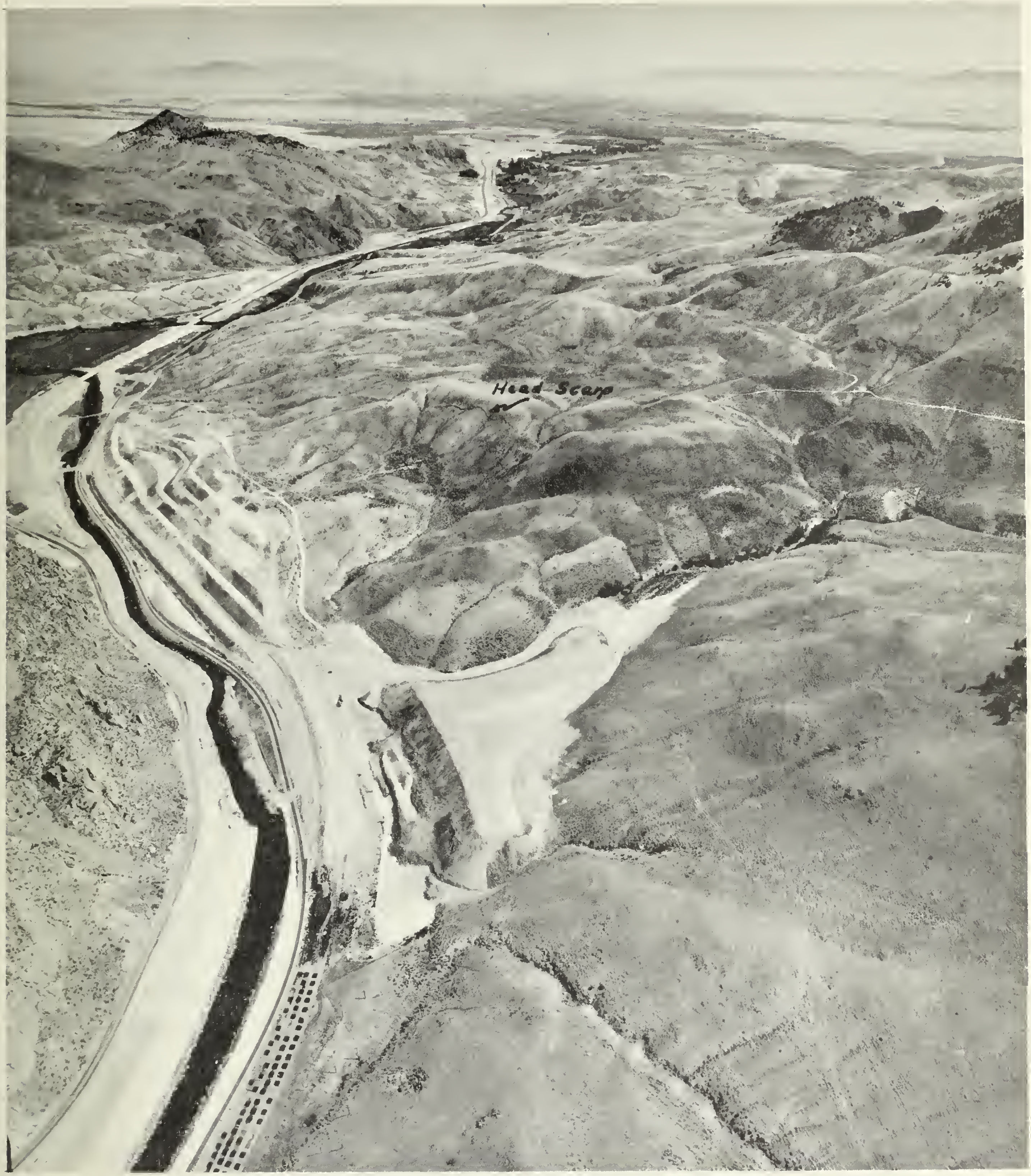


Figure 3-2, Oblique photograph taken July 5, 1966 looking to the northeast, showing the complete area of this ancient landslide complex. A portion of another landslide complex, of equal size, can be seen in the upper left hand corner of this photograph. The head scarp and the northern boundary of the present slide are located in the left center portion of this photograph. The railroad backslope was designed as a corrective measure against the 1965 landslide, this slide was located in the area where the benches have been widened. The fill in the center of the photograph is the waste pile from the railroad cut section.

LANDSLIDE NO. A-11

Pipe Organ Landslide

HISTORY

The Pipe Organ Landslide is located near Dalys Spur approximately 13 miles south of Dillon, Montana. The first major movement occurred during the construction of a cut section for the relocation of the Union Pacific Railroad during the 1965 construction season. Since the first movement two additional independent landslides have occurred in this cut section. The second and most serious movement occurred upon the completion of the cut section in the early spring of 1966 and initiated a detailed investigation of this landslide area. The third landslide is a relatively minor movement that occurred in the spring of 1967 and is a cut slope failure located between the first and second slides.

The original proposed design for Interstate 15 required that the Union Pacific Railroad and the Beaverhead River be relocated into the east side of the Beaverhead Valley to provide for the I-15 alignment.

All of the slides are located in an ancient landslide complex which is roughly 4800 feet wide at the toe and 4400 feet in length. The removal of a portion of the toe of this ancient landslide mass through the above construction for the railroad triggered these slides. Figure 3-2, page 91, illustrates all of the above features with the exception of the third movement.

The movement of 1965, located between station 9630± and 9636± of the railroad relocation line, (see figures 3-1 and 3-2, pages 90 and 91), was investigated and stabilized. This slide was essentially a rotational movement with a relatively shallow slide plane, approximately 30 feet in depth. The investigation included two vertical drill holes, electrical resistivity survey and stability analysis. This investigation led to the remedial action of redesigning the cut slope from a 2:1 cut to a slope with a series of benches resulting in a composite slope of approximately 4:1. The remaining cut slope to the south was also redesigned with benches to make a composite slope of

approximately 3:1, figure 3-2, page 91, illustrates this design. This corrective measure removed the sliding material in the first failure and this area has remained in a stable condition.

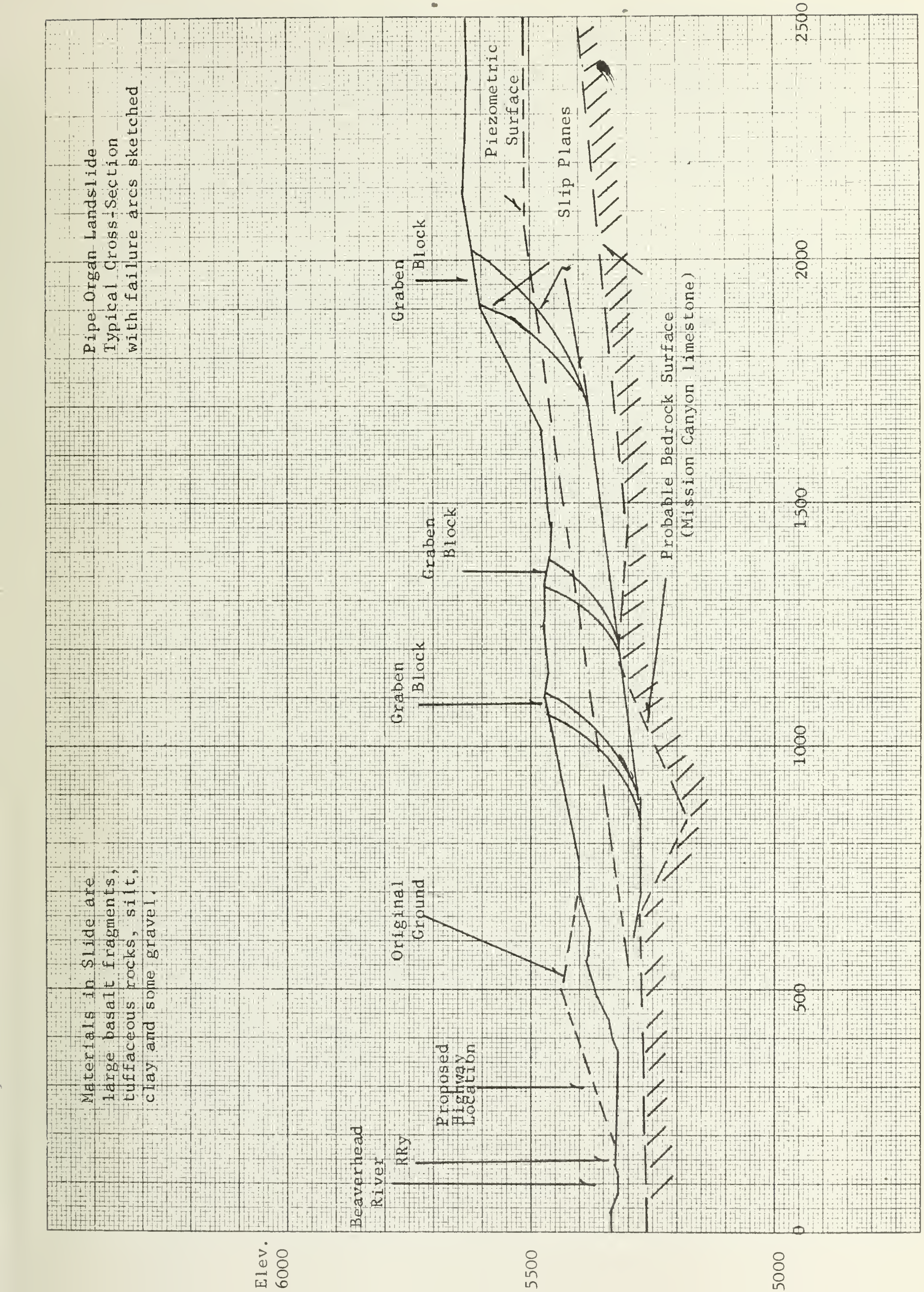
The second and most serious slide, is located between Stations 9616± and 9626± of the railroad relocation line. This landslide can be best described as very complex with failure being classified as a combination of planar and rotational movements. (Plate 2 on page 94 is an idealized cross section parallel to slide movement and through the center of the slide). The dimensions of this slide are 1800 feet in length and 800 feet in width at the head and 1200 feet in width at the toe. This slide has a maximum observed depth of 160 feet with an average depth of 110 feet. Approximately nine million cubic yards of material are involved in this slide. The development of this slide instigated a detailed investigation to determine the possible corrective measures to complete the Interstate location. The data obtained and the results of this investigation are presented in some detail in the remainder of this report.

The third landslide occurred during the summer of 1967 and is located between Station 9627± and 9631± of the railroad relocation line. This landslide is a rotational slide and is very similar in size and classification as the first landslide of 1966. This slide has been mapped and observed for slide movement; however, corrective measures are not planned until the major landslide can be stabilized.

Physiography and Climate

The landslide area is located in portions of Sec. 36, T-8S, R-10W, and Sec. 31, T-8S, R-9W, Beaverhead County, Montana. The area is at the west end of the Blacktail Mountain Range and is bounded on the west by the Beaverhead River. The area is a moderately steep, westward facing slope, typically hummocky, with elevations ranging from approximately 5320 feet near the river to 6,589 feet on top of the ridge that forms the eastern edge of the slide area.

The climate consists of cool summers and cold winters. The average annual precipitation at Western Montana College of Education at Dillon for 30 years (1930 to 1960) is 11.3 inches and the average annual precipitation at the Dillon airport for 21 years (1939 to 1960) is 9.1 inches. Since the slide area is at a


Pipe Organ Landslide
Typical Cross-Section
with failure arcs sketched
Materials in Slide are
large basalt fragments,
tuffaceous rocks, silt,
clay and some gravel.
Beaverhead River
RRy
Proposed Highway Location
Original Ground
Graben Block
Graben Block
Graben Block
Piezometric Surface
Slip Planes
Probable Bedrock Surface
(Mission Canyon limestone)
Elev.
6000
5500
5000
0
500
1000
1500
2000
2500

higher elevation than these weather stations, it is estimated that the annual precipitation at the slide area is from 10 to 16 inches, with the upper portion of the slide area receiving more precipitation then the lower portion. Surface drainage of the slide area is confined to intermittent streams, all of which are tributary to the Beaverhead River.

GEOLOGY

The specific area of the landslide contains from 40 to over 240 feet of volcanic and alluvial materials, which at one time were fairly uniform layers of ash, tuff, rhyolite and basalt flows of Tertiary age that are now reduced to a heterogeneous mass that can be best described as rubble. Large blocks of basalt and rhyolite are completely isolated in a mass of decomposed or partially decomposed ash and tuff. Decomposition of the ash has resulted in large zones of bentonitic clays. Tuffaceous beds have been ground and decomposed to silty clay. Columnar structure within the basalt which was originally vertical is visible near the toe of the slide inclined 15° from horizontal. The Tertiary volcanics in the portion of the slide area near the river are underlain by the Quadrant Quartzite and it is presumed that the remainder of the volcanics east of the river are underlain by limestone of the Madison Group. Vuggy limestone, probably from the Mission Canyon formation of the Madison Group, was penetrated in test holes D and W. A high angle normal fault can be observed in a small drainage to the south at the toe of the present slide. The brecciated quartzite in the fault zone was penetrated in Test Hole No. 1.

The original landslide complex is postulated to have been caused by a combination of over-steepening of the valley walls by the Beaverhead River, weakening, by ground water, of the softer volcanic rock units, and perhaps a seismic disturbance of severity. The evidence indicates that since the original movement many major movements have occurred, see figure 3-1 and 3-2, pages 90 and 91. The landslide appears to be confined to Tertiary deposits.

The ridge forming the eastern edge of the slide is primarily composed of highly fractured and broken Tertiary volcanic flow rocks of rhyolite and basalt compositions.

During the initial subsurface exploration, large amounts of ground water were encountered within the sliding mass. The presence of the ground water was considered a basic problem and, therefore, the investigation was expanded to study the ground water in the immediate area to determine the nature of the aquifer and the possibilities of controlling or dewatering the slide. Based on general hydrological evidence it appears that the ground water in this slide area originates from local infiltration of precipitation and is not a part of a large regional ground water system. A series of pump tests in the area found that (1) water can be controlled by pumping within the slide, (2) removal of ground water directly affects the slide movement, (3) the ground water is hydraulically disconnected into discreet blocks which probably reflect present and past slide blocks.

A pump system has been installed, with pumps located in all the major block segments of the slide to remove the ground water. According to monthly observations of movement control points the slide has not moved for the past 2 years.

In addition to the pumping system other remedial action that has taken place includes the following:

(1) Establishment of surface drainages throughout the slide-complex area.
(2) Establishment of permanent control markers to observe slide movement.
(3) Periodically sealing of all the surface cracks to avoid further saturation of the slide plane by precipitation.
(4) Relocation of I-15.

Further stabilization can be obtained, by the construction of a counterbalance berm in the area that was originally to be the railroad centerline. A strut type of feature in the toe area that would require sub-excavation of 20 feet ± of slide rubble and enable the berm to key into dense gravel at this depth has also been considered.

Due to the extensive amount of time and work that has gone into this project and the resulting abundance of information, a more detailed discussion of the investigation is not possible in the space allotted.

Pipe Organ Slide

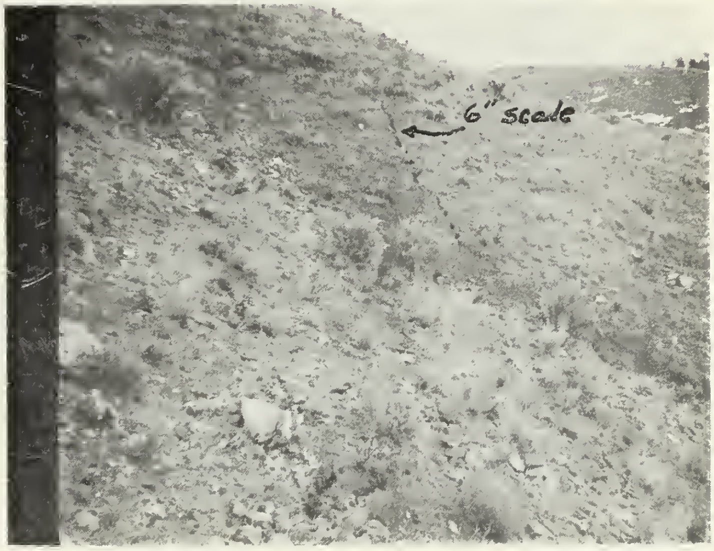


Figure 3-3, View looking south at the head scarp of the present slide, date, March 16, 1966. A six inch scale is placed across the scarp in the upper center portion of this photograph.

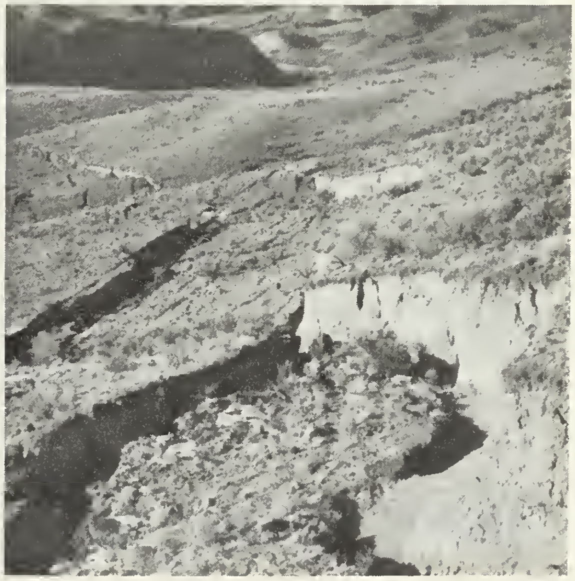


Figure 3-4, View looking northwest of the head scarp of the present slide, photograph date, January 12, 1967.

Pipe Organ Slide

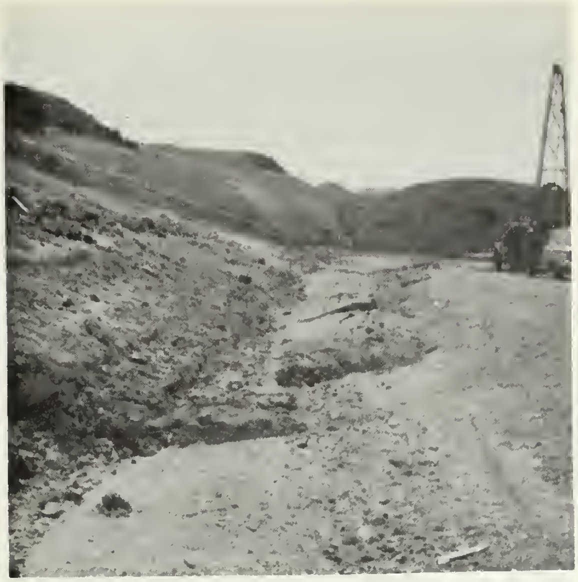


Figure -5, View looking south of the toe development of the present slide. The first expression of the toe of this slide was noted in the above Railroad relocation ditch-line. Photo date, May 24, 1966.

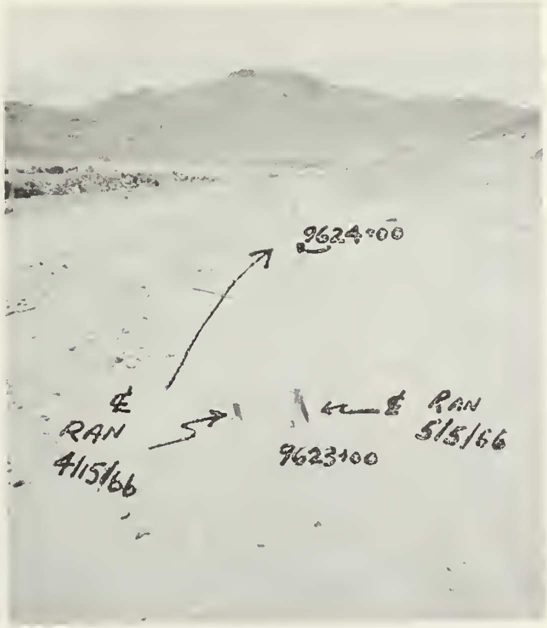


Figure 3-6, View looking north and along centerline of the Railroad relocation line. The double stakes clearly show the movement that took place between the two surveys. Note the large landslide feature in the background. Photo date, May 5, 1966.

Pipe Organ Slide

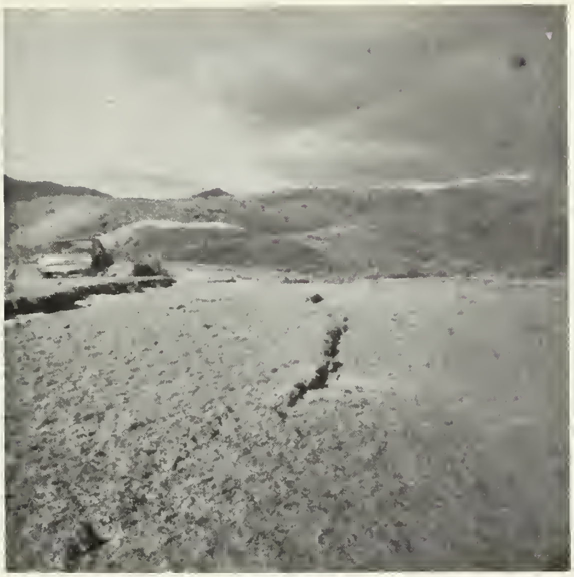


Figure 3-7, View looking north at the approximate center of the present landslide. Photo date, January 12, 1967.

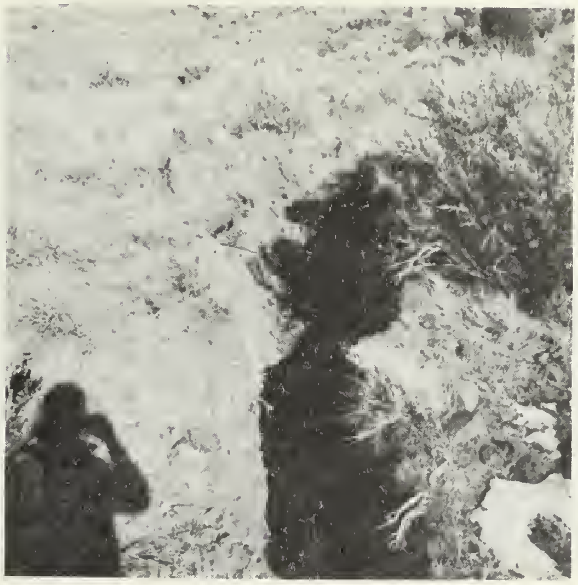


Figure 3-8, View looking east along the south side of the slide. This crack is approximately 2.0 feet wide and essentially vertical for 10 to 15 feet. Photo date, January 12, 1967.

Pipe Organ Slide

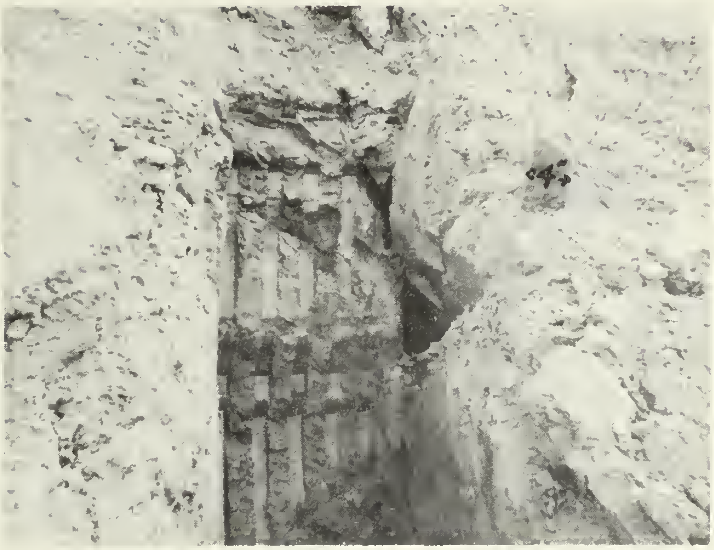


Figure 3-9, Photograph of trench dug to obtain undisturbed sample near the slide plane, date March 16, 1966. The dark areas at the upper right side of the trench is a part of the present slide plane. Many old slide surfaces are visable in this trench. The block of material used for a scale shows two of these old shear planes.

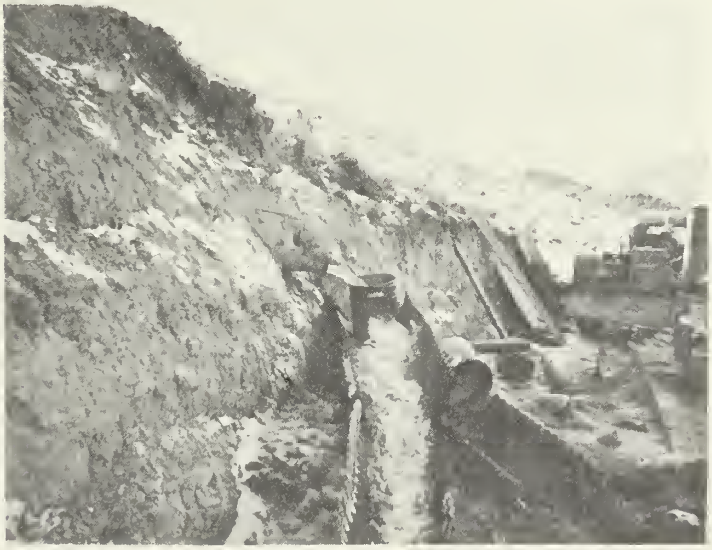
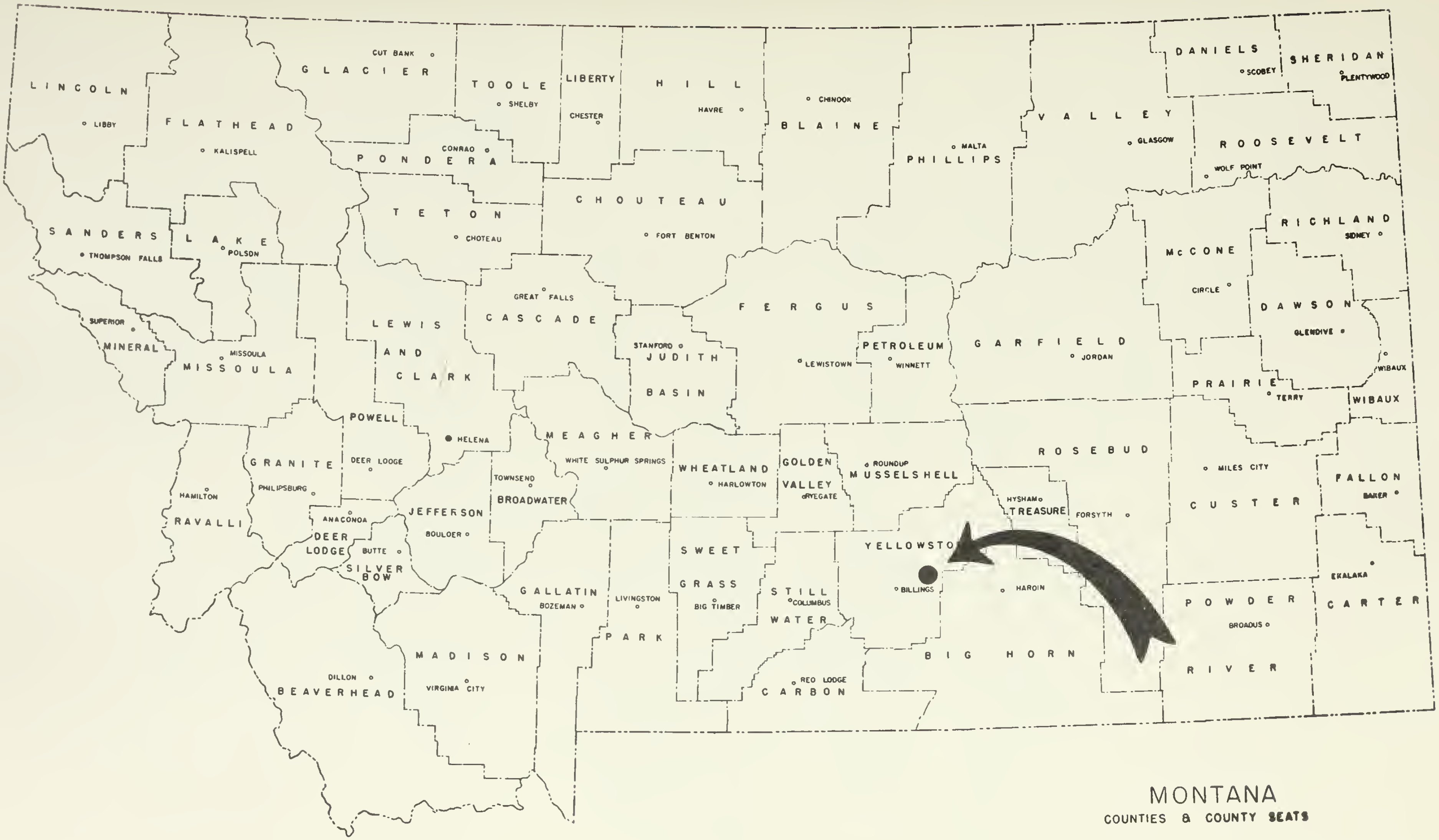


Figure 3-10, View looking north of horizontal drill hole # 2 rate of flow 150 gpm, before hole plugged by caving.

BALLANTINE LANDSLIDE



SLIDE NUMBER A-12

Landslide No. A-12

Ballantine Landslide

Description of Failure

In December 1967, while constructing a portion of Interstate 94 east of Billings, a failure developed in a cut slope section which had just been completed.

The materials involved were Bearpaw shale of Cretaceous age overlain by 15 to 20 feet of terrace gravel. Ground water was present in large quantities at the materials interface because of the topography and also because of return water from irrigation. (This was a major source during the irrigating season as evidenced by rises in the ground water table in the vicinity during summer months.)

The Bearpaw formation is a poorly indurated marine clay shale which exhibits the characteristics of a soil classified as A-7-6 shortly after exposure to weathering. In addition, bentonite seams are usually present in layers to 3 feet in thickness.

First inspection of the failure disclosed an open tension crack approximately 250 feet right of the cut face, extending 400 feet along the hillside. Toe development had not progressed beyond a slight outward bulge in the cut face. Several active springs were noted within the failure zone and a program of exploratory drilling was initiated. The topography indicated that a pre-existing landslide was involved.

Sub-surface Exploration

Drilling disclosed a high water table and pump tests indicated that a permeable zone existed which could be drained. Horizontal drilling was initiated immediately which adequately drained the excess water, lowered the hydrostatic head and stabilized the area.

Stability Analysis

The stability analysis which follows indicated the advisability of drainage. The shearing resistance obtained from Laboratory Testing of triaxial shear samples recovered during the drilling program was used directly. The hydrostatic gradient had been established during drilling and the seepage force was readily calculated.

Materials testing resulted in the following average values for the disturbed shale:

Angle of friction = 10^{o}
Cohesion = 240 lbs/ft.^{3}
In-Place density = 130 lbs/ft.^{3}

Shearing strength of the overlying gravel was considered to be:

Cohesion = 0 and friction angle = 25^{o}.

The following equation was used in the analysis:

$$\text{Factors of safety} = \frac{cl + (N-u)\ \text{Tan}\ \emptyset}{T + \text{Seepage force}}$$

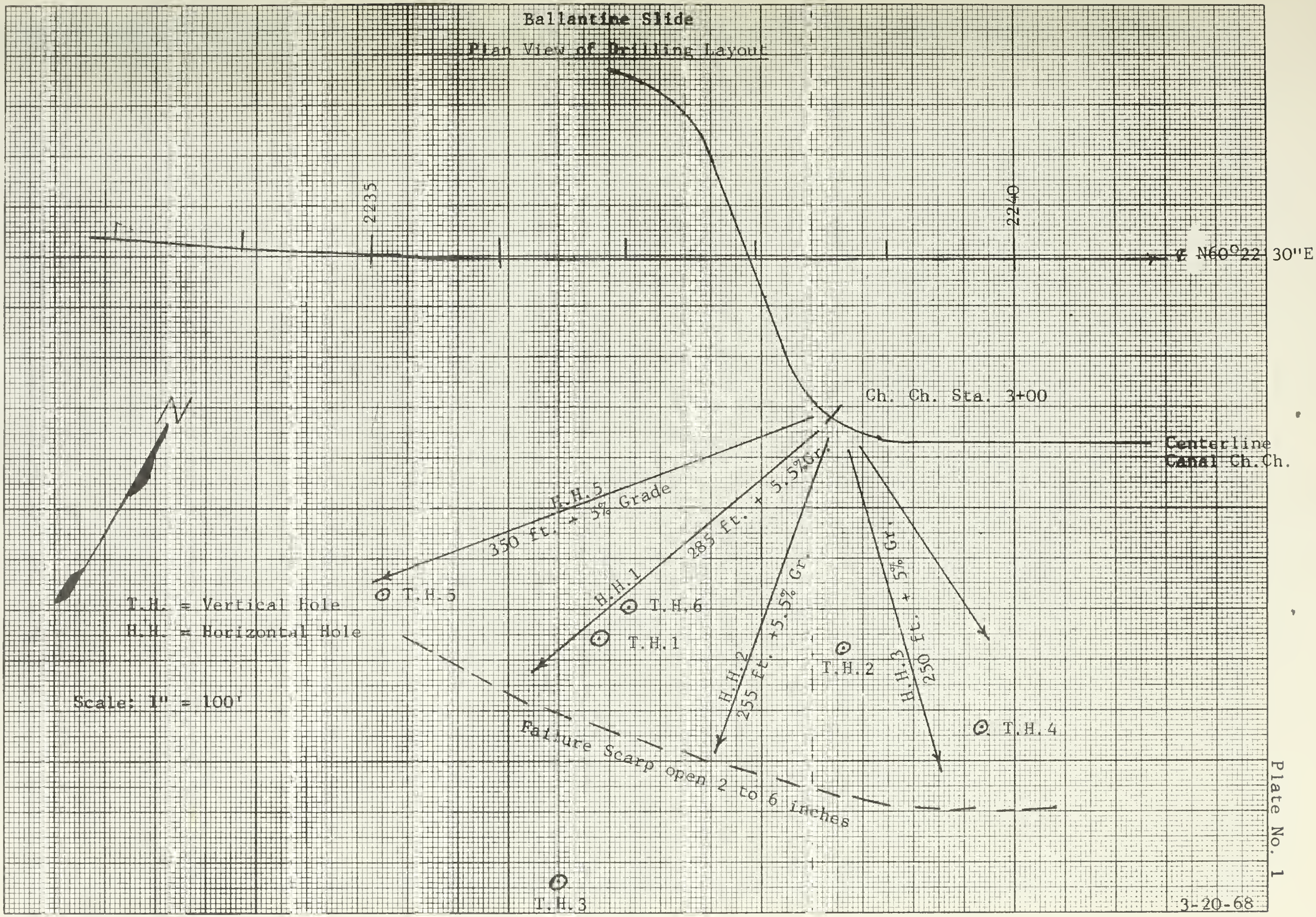
where c = Cohesion
l = length of failure surface
N - force normal to failure surface
u = force of soil water acting upward on the failure surface
Ø = angle of friction
T = force tangential to failure surface

Analysis of the critical arc depicted on Plate No. 2, page 105, resulted in the following:

Condition prior to construction, SF = 1.09
Condition after construction, SF = 0.85
Condition with horizontal drains, SF = 1.24
Condition with horizontal drains and cut slope flattening, SF = 1.22
Condition with horizontal drains and head unloading to a depth of 20 feet, SF = 1.50

Inasmuch as the factor of safety in the condition immediately after construction was probably nearer 0.95 than 0.85, the factor of safety obtained with the horizontal drains alone was considered adequate.

Plate No. 1



Ballantine Slide

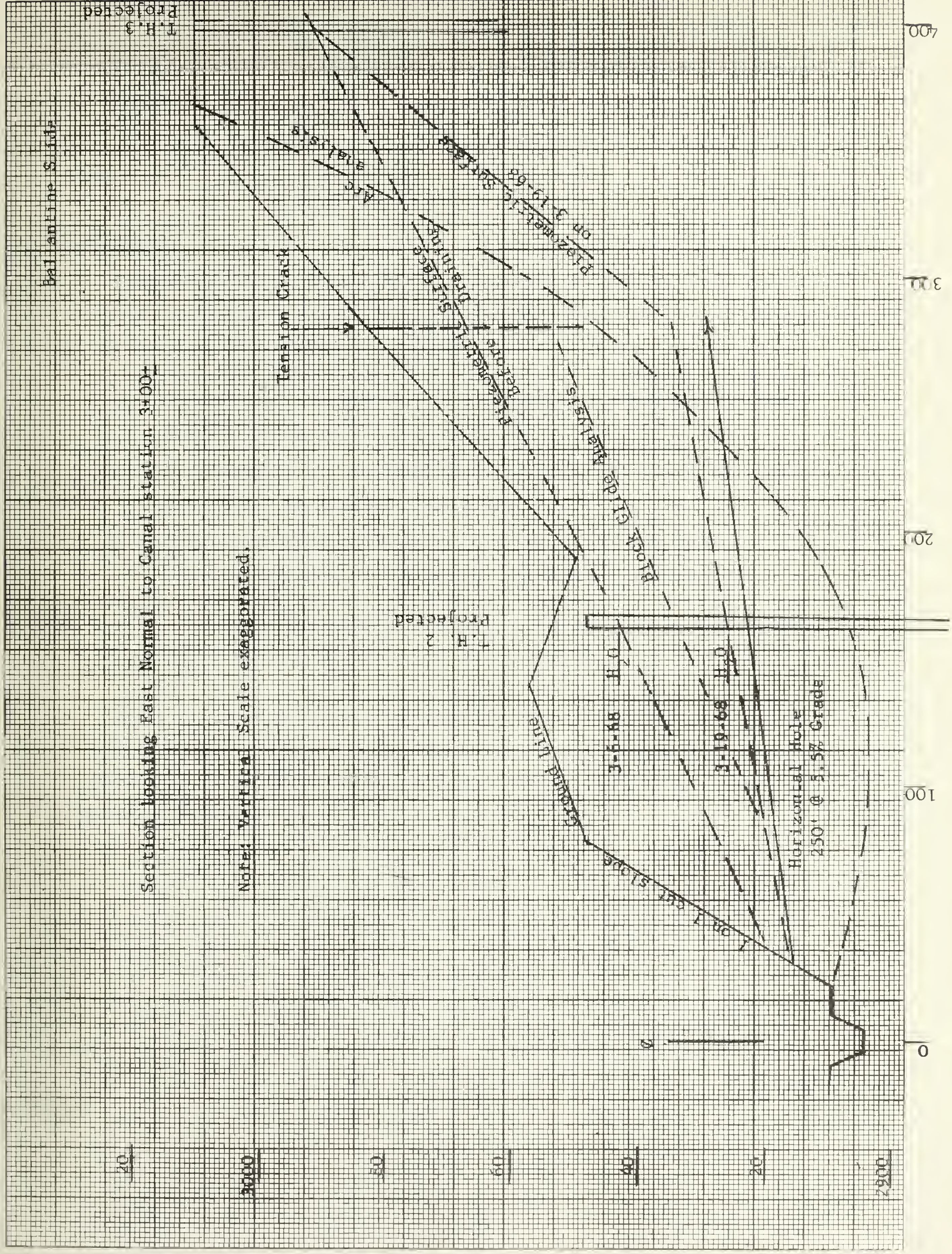
Plan View of Drilling Layout

T.H. = Vertical Hole
H.H. = Horizontal Hole

Scale: 1" = 100'

3-20-68

Plate No. 2


Ballantine Slide
Section Looking East Normal to Canal station 3+00±
Note: Vertical Scale exaggerated.
T.H. 3 Projected
T.H. 2 Projected
Arc analysis
Piezometric surface on 3-19-68
Tension Crack
Surface Drainage
Before Piezometric
Block Glide Analysis
Ground line
1 on 1 cut slope
3-6-68 H2O
3-19-68 H2O
Horizontal Hole 250' @ 3.5% Grade
℄
400
300
200
100
0
20
3000
80
60
40
20
2900

Remedial Action

Page 105 illustrates the immediate response to drainage accomplished. The drains continued to flow at approximately 20 to 30 gpm total through the summer.

Discussion

This landslide is one of the best examples in Montana which could be corrected by horizontal drain holes as the conditions which control such drainage are critical. These conditions include the following:

(1) An existing hydrostatic head and gradient which contributes greatly to the unstable condition.

(2) A permeable zone of sufficient magnitude and depth that drainage will alter the unstable condition.

(3) A platform to drill from which is at such an elevation as to allow drainage and near enough to the aquifer to allow drilling.

(4) A drilling method which guarantees the drill hole will remain intact until the drain pipe is installed.

This landslide re-activation could possibly have been prevented by line or grade adjustment, however, the unstable condition would still be a menace. It would have been difficult or impossible without drilling to predict the conditions which existed.

(4) CONCLUSIONS

All landslides investigated were related to the geologic setting and several general conclusions can be drawn.

(1) The clay shale units of Jurassic through Tertiary age are susceptible to failure when allowed to become saturated. Particularly sensitive are the Morrison fm of Jurassic age, the Cretaceous age Colorado shale units, the Bearpaw shale of upper Cretaceous age and the soft shales of the Tertiary age Ft. Union fm. The lake bed sediments of Tertiary and Pleistocene age and glacial till of Pleistocene time are susceptible to failure also. Wherever these units are encountered, special investigation should be made to determine stability.

(2) The geologic structure of the rocks determine to a large extent the stability of proposed cuts or fills. The attitude of bedding and jointing is very important, as is the weathering characteristics of the mineral constituents of the rock.

(3) Ground water contributed to the failure of the great majority of landslides studied in some form or other. Elimination of this water will, in many cases, stabilize the area. This removal is dependent upon the permeability of the materials, and is often possible within economic limitations.

(4) Landslides and potential failure areas are predictable in most cases, if investigation is made in time to allow proper analysis of the area in question.

(5) Although not complete, our library will provide aid in evaluating the stability of a given area with a minimum of investigation.

(5) FURTHER RESEARCH RECOMMENDATIONS

(1) Evaluation should be made for the applicability of more sophisticated instrumentation to landslide investigation and/or stability problems of all types.

a. In-place shear strength testing equipment.

b. Slope indicators to determine the amount and direction of movement in the drilled holes.

c. In-place density and moisture determination through drill casing.

d. Use of colored, infra-red or other photography.

(2) Development of a computer program should be possible into which the raw data such as the location and elevation of surface control points, physical characteristics of materials involved, surface and ground water conditions, safety factors desired as well as those assumed from the condition of the failure, etc. can be inserted. The program could possibly be set up as a mathematical three dimensional model and submit the results which could be expected by drainage, toe loading, head removal, or combinations thereof. The results obtained could then be easily reduced to determination of the most desirable solution from an economic standpoint.

REFERENCES

Billings Geological Society 1952 to 1957 and 1962, GUIDE BOOKS, Annual Field Conferences, Billings, Montana

Cleaves, Arthur B. 1961 "LANDSLIDE INVESTIGATIONS -a field handbook for use in highway location and design." U.S. Department of Commerce, Bureau of Public Roads. U.S. Government Printing Office, Washington, D. C.

Jakosky, J. J. 1950 "EXPLORATION GEOPHYSICS" Trija Publishing Company, Los Angeles, California

Krynine, D. P., and Judd, W. R. 1957 "PRINCIPLES OF ENGINEERING, GEOLOGY AND GEOTECHNICS" McGraw-Hill Book Company, Inc. New York, N. Y.

Montana Bureau of Mines and Geology, "MEMOIRS", "BULLETINS", "SPECIAL PUBLICATIONS", and "SPECIAL REPORTS", Montana College of Mineral Science and Technology, Butte, Montana

National Research Council 1958 "LANDSLIDES AND ENGINEERING PRACTICE". Highway Research Board Special Report 29. NAS-NRC Publication 544

Ross, C. P., Andrews, D. A. and Witkind, 1955, "GEOLOGIC MAP OF MONTANA" U.S.G.S. Washington, D. C.

Sowers, G. B., and Sowers, G. F. 1951 "INTRODUCTORY SOIL MECHANICS AND FOUNDATIONS". The Macmillan Co., New York, New York

Taylor, D. W. 1948 "FUNDAMENTALS OF SOIL MECHANICS" John Wiley & Sons, New York, New York

Terzaghi, Karl, and Peck, R. B. 1948 "SOIL MECHANICS IN ENGINEERING PRACTICE" John Wiley & Sons, New York, N. Y.

Terzaghi, Karl 1950 "MECHANISM OF LANDSLIDES" in "APPLICATION OF GEOLOGY TO ENGINEERING PRACTICE" Berkley Volume, Sidney Paige, Chairman; Geol. Soc. America, p. 83-123

Terzaghi, Karl 1960 "FROM THEORY TO PRACTICE IN SOIL MECHANICS" John Wiley & Sons, Inc., New York, New York

U. S. Geological Survey, "TOPOGRAPHIC MAPS","GEOLOGICAL MAPS", "BULLETINS", "WATER SUPPLY PAPERS", and "PROFESSIONAL PAPERS", United States Government Printing Office, Washington, D. C.

Wu, T. H. 1966 "SOIL MECHANICS" Allyn and Bacon, Inc., Boston, Mass.